CIVIL ENGINEERING

PRESTRESSED

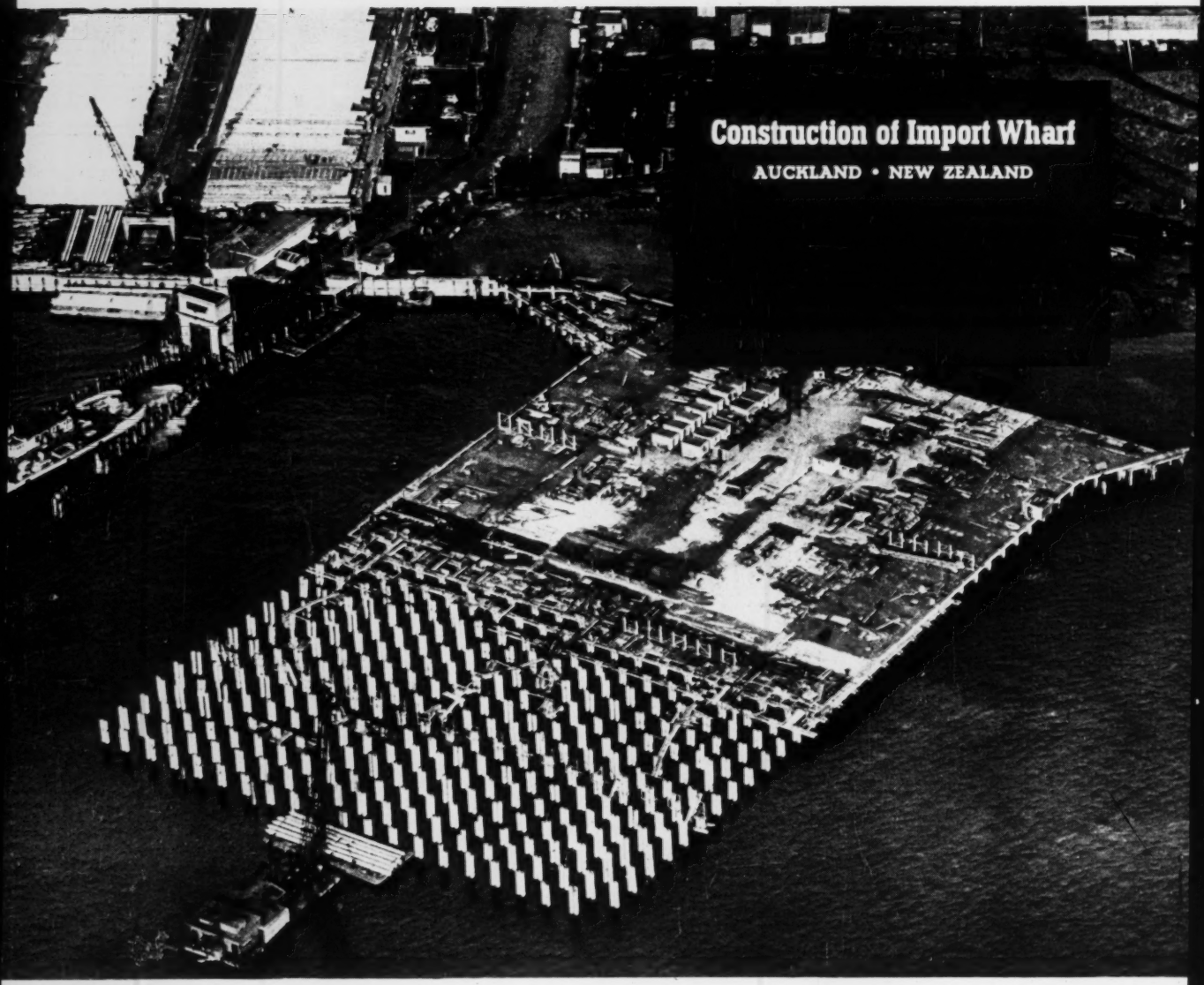
CONCRETE

PILES

See article by Maxwell Upson

PRESTRESSED CONCRETE ISSUE

DOWN UNDER with RAYMOND



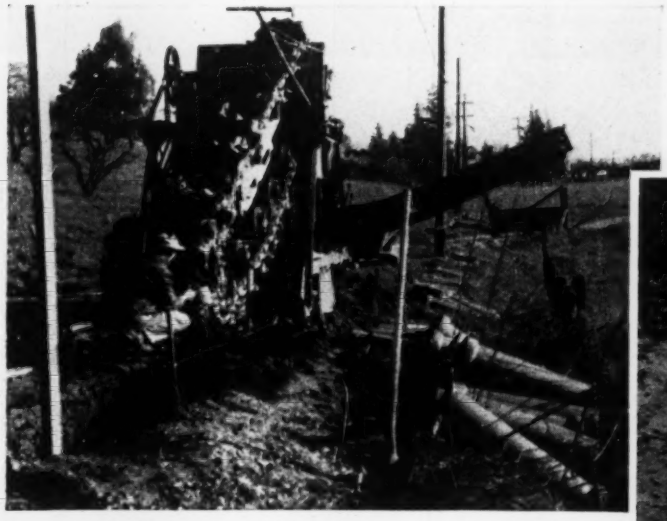
A Raymond-supervised organization is constructing an ocean freight terminal in Auckland, which includes a large reinforced concrete pier of earthquake-resistant design, equipped with rail, warehouse and auxiliary facilities. When completed by mid-1953, the new facilities, providing berthing space for four big ocean freighters, will increase the capacity of the port by some 550,000 tons annually . . . another outstanding example of Raymond's service to the ports of the world.

RAYMOND CONCRETE PILE CO.

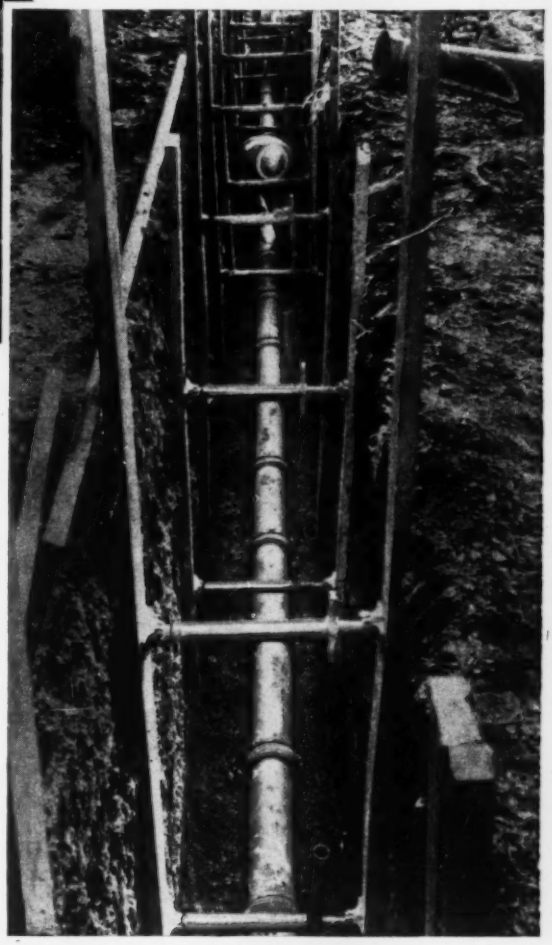
140 CEDAR STREET • NEW YORK 6, N.Y.

THE SCOPE OF RAYMOND'S ACTIVITIES . . . *FOUNDATION CONSTRUCTION . . . SOIL INVESTIGATIONS . . . IN-PLACE PIPE LINING . . . HARBOR AND WATERFRONT IMPROVEMENTS . . . SPECIALIZED CONSTRUCTION*

CLAY PIPE—*ESSENTIAL* ★ *ECONOMICAL* ★ *EVERLASTING*



Clay Pipe—381,000 feet of it—was used in this sewerage project northeast of San Jose, California, a residential area that mushroomed as defense workers poured into the area.



CLAY PIPE gets another 72-Mile Testimonial

Here's a testimonial 381,000 feet long, proving once again that Vitrified Clay Pipe is the time-tested material engineers and public officials depend on for sanitary sewerage. Thanks to their foresight, thousands of new homes in the San Jose district, as well as essential housing projects throughout the nation, will enjoy the *permanent* sanitary protection that only clay pipe can offer.

Vitrified Clay Pipe assures generations of trouble-free service. Acid sewage or alkaline soils can't corrode it . . . time can't weaken it. It's the only chemically inert sewerage material — *the only pipe that never wears out!*

NATIONAL CLAY PIPE MANUFACTURERS, INC.

1520 18th St. N. W., Washington 6, D. C.
206 Connally Bldg., Atlanta 3, Ga.
100 N. LaSalle St., Rm. 2100, Chicago 2, Ill.
703 Ninth & Hill Bldg., Los Angeles 15, Calif.
311 High Long Bldg., 5 E. Long St., Columbus 15, Ohio

WHEREVER RELIABLE PERFORMANCE-PROVED PIPE IS NEEDED, SPECIFICATIONS CALL FOR VITRIFIED CLAY

South Bend, Ind. (Private Housing)	93,000 ft.
Tuscaloosa, Ala. (Municipal Expansion)	44,000 ft.
Evansville, Ind. (Municipal Expansion)	200,000 ft.
Detroit, Mich. (Jet Engine Plant)	28,000 ft.
Topeka, Kan. (Private Housing)	13,000 ft.
Tucson, Ariz. (Municipal Expansion)	38,000 ft.
Minneapolis, Minn. (Private Housing)	80,000 ft.

Vitrified
CLAY
CLAY PIPE NEVER WEARS OUT!
PIPE

C-1252-59

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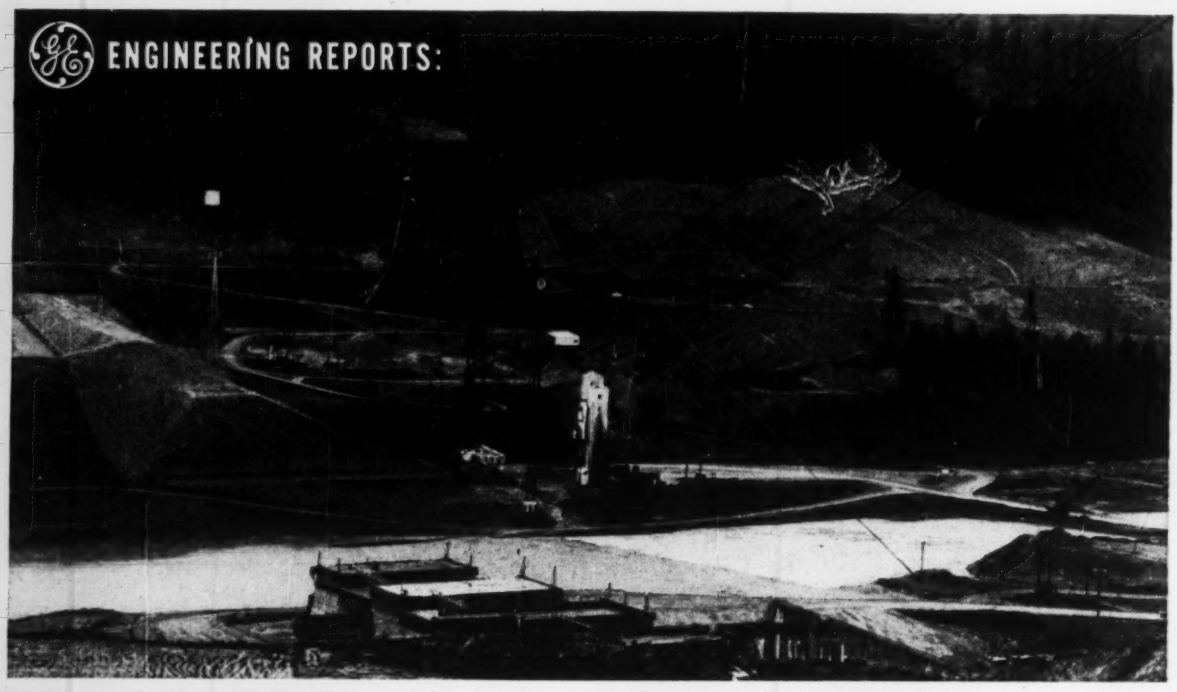
ENGINEERING REPORTS:



800,000 YARDS OF CONCRETE will be needed to complete Lookout Point Dam on the Willamette River near Lowell, Oregon. G-E powered cableway—2600 feet long with 25-ton hook capacity—is of radial type, with stationary 428-foot-high head tower and movable tail tower. Maximum conveying speed is 1800 fpm, maximum hoisting speed is 470 fpm.

Complete electrification keeps Lookout Point on schedule

Morrison-Knudsen selects G-E drives to batch and place concrete at Oregon dam

To meet a tight work schedule at the Corps of Engineers' Lookout Point Dam, the Morrison-Knudsen Co. relies on modern construction equipment—electrified throughout by General Electric. This equipment, spearheaded by a Travelift cableway built by Construction Improvements Ltd., and Lidgerwood Industries Inc. is helping to place an average of 2000 yards of concrete per day.

Lookout Point, an earth-and-gravel-fill dam located at Lowell, Oregon, is a combined flood-control and power-generation project scheduled for 1953 completion. To heavy construction people, it provides one more example of the effective use of *electrified* equipment on jobs where smooth, dependable operation is an everyday "must."

Whether you buy or build construction equipment, you'll find it's safer, more flexible, and more efficient when electrified by G.E.—with skilled engineering help in application, installation, and service. Find out how this equipment and service can pay off for you by contacting your nearest G-E Apparatus Sales Office. General Electric Co., Schenectady 5, N. Y. 664-25

CABLEWAY is driven by this G-E 500-hp, 2300-volt wound-rotor motor. G-E drives also power conveyors, rock crushers, and batch-plant equipment.

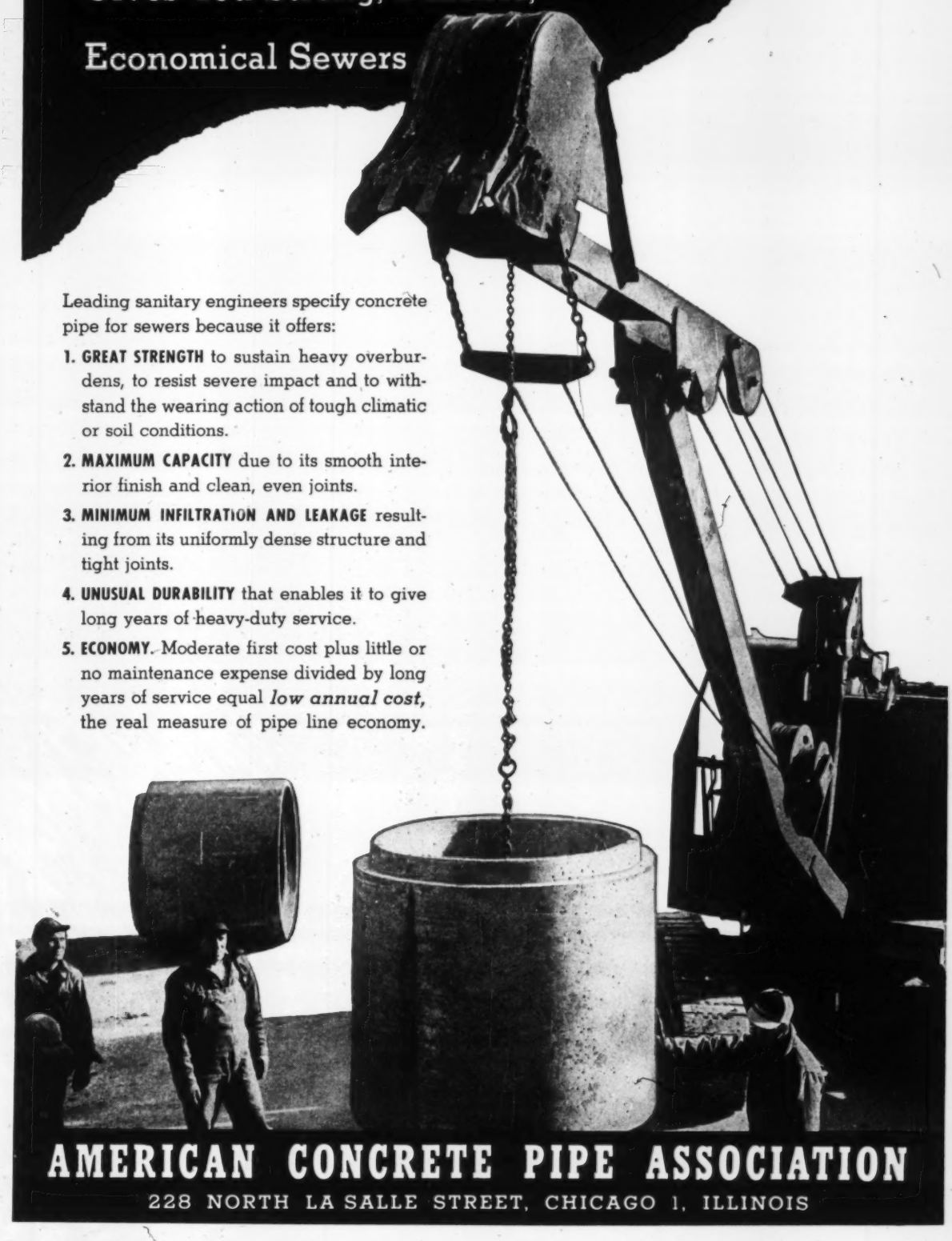
CENTRALIZED CONTROL equipment for cableway includes (left to right) Limitamp high-voltage primary panel, secondary control, and cast-grid resistors.

Engineered Electrical Systems for Heavy Construction

GENERAL ELECTRIC

CONCRETE PIPE

Gives You Strong, Durable, Economical Sewers

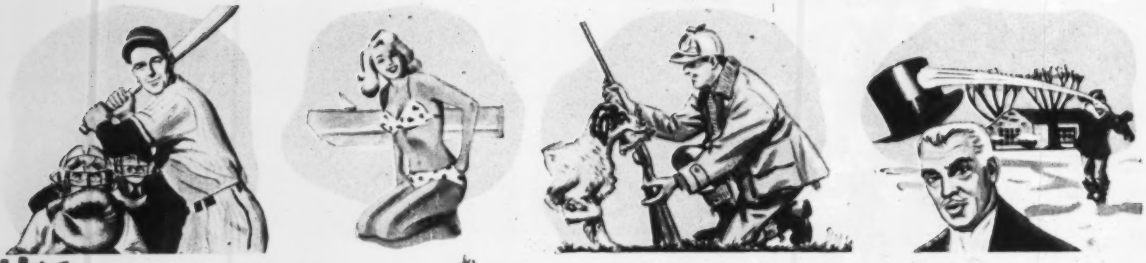


Leading sanitary engineers specify concrete pipe for sewers because it offers:

1. **GREAT STRENGTH** to sustain heavy overburdens, to resist severe impact and to withstand the wearing action of tough climatic or soil conditions.
2. **MAXIMUM CAPACITY** due to its smooth interior finish and clean, even joints.
3. **MINIMUM INFILTRATION AND LEAKAGE** resulting from its uniformly dense structure and tight joints.
4. **UNUSUAL DURABILITY** that enables it to give long years of heavy-duty service.
5. **ECONOMY.** Moderate first cost plus little or no maintenance expense divided by long years of service equal *low annual cost,* the real measure of pipe line economy.

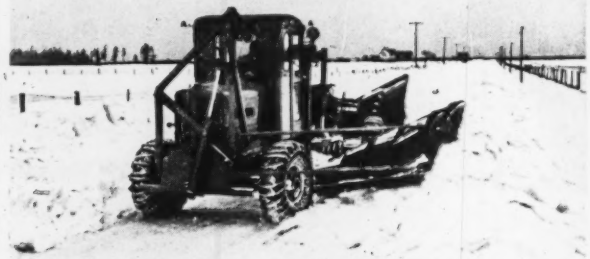
AMERICAN CONCRETE PIPE ASSOCIATION

228 NORTH LA SALLE STREET, CHICAGO 1, ILLINOIS



LIKE Spring and Baseball...Summer and Swimming...Autumn and Hunting...Winter and Snowballs

They Go Together All Year 'round



All Wheel Drive and All Wheel Steer

Yes, whether it's pulling a wet ditch, with the rear drivers up where the footing is good; or finishing a wide shoulder without leaving tire marks; or reaching out for a tremendous windrow and missing it with all wheels; or steering the rear wheels against the side thrust when widening out... whatever the season... whatever the job... All-Wheel Drive and All-Wheel Steer work together as a team... each making the other just that much more effective.

NO TWO WAYS ABOUT IT . . . an Austin-Western Power Grader goes places where ordinary motor graders cannot go... does things they cannot do... saves time and money on every job.

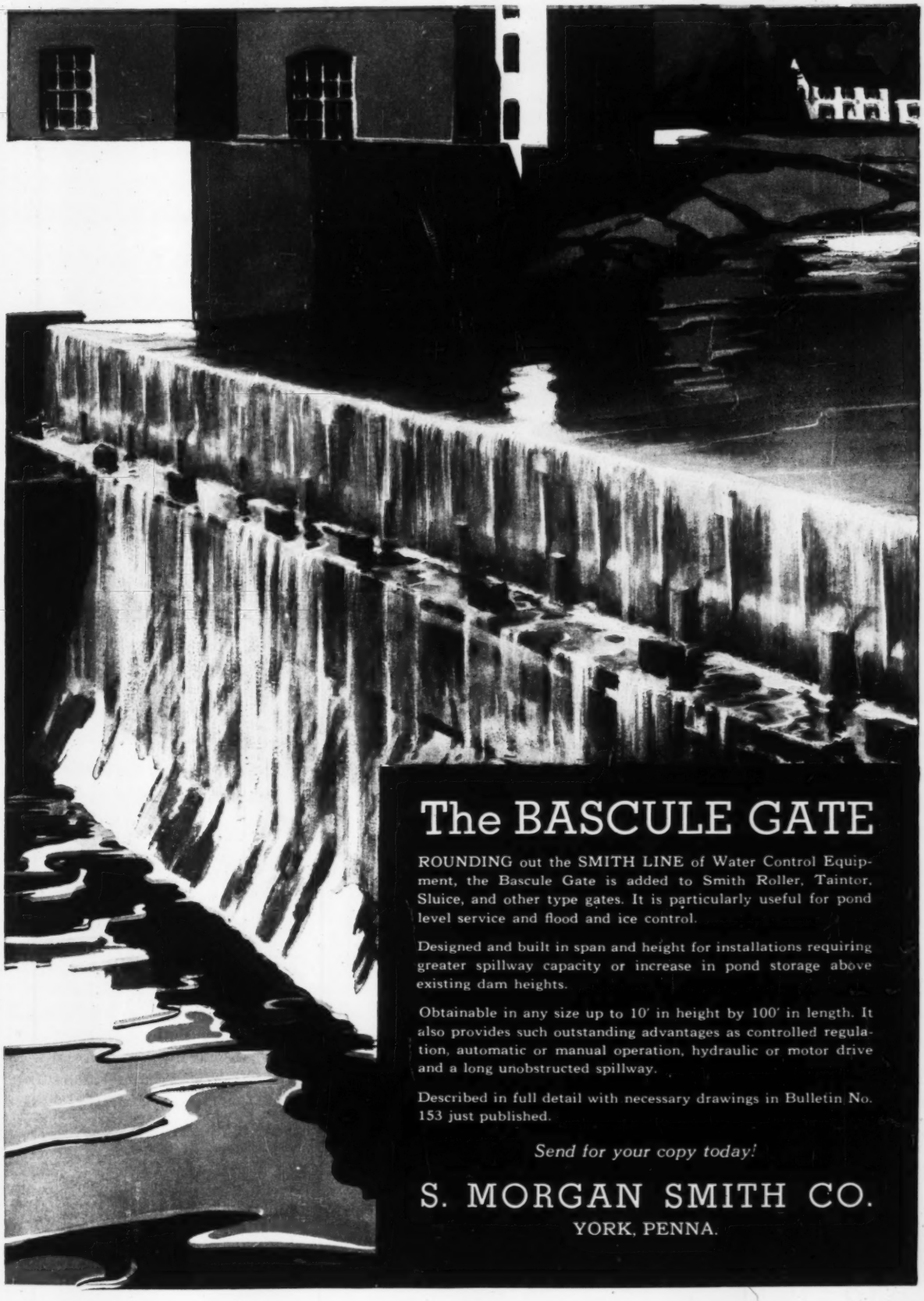
Austin-Western

Power Graders
Road Rollers · Motor Sweepers

BALDWIN BLH LIMA-HAMILTON

Construction Equipment Division

Manufactured by
AUSTIN-WESTERN COMPANY
Subsidiary of Baldwin-Lima-Hamilton Corporation
AURORA, ILLINOIS, U.S.A.



The BASCULE GATE

ROUNDING out the SMITH LINE of Water Control Equipment, the Bascule Gate is added to Smith Roller, Taintor, Sluice, and other type gates. It is particularly useful for pond level service and flood and ice control.

Designed and built in span and height for installations requiring greater spillway capacity or increase in pond storage above existing dam heights.

Obtainable in any size up to 10′ in height by 100′ in length. It also provides such outstanding advantages as controlled regulation, automatic or manual operation, hydraulic or motor drive and a long unobstructed spillway.

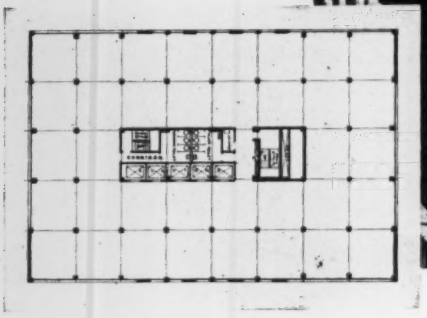
Described in full detail with necessary drawings in Bulletin No. 153 just published.

Send for your copy today!

S. MORGAN SMITH CO.

YORK, PENNA.

Below: Typical floor layout, showing reinforced concrete central core and exterior panels which effectively resist the horizontal loads developed by earthquake forces.



Architect: Claude Beelman; *Associate Architect:* Herman Spackler; *General Contractor:* C. L. Peck; *Structural Engineers:* Brandow and Johnston.

LOS ANGELES OFFICE BUILDING

Has Simple, Light Steel Frame

This 12-story, 3-unit office building in Los Angeles, built recently for the Tishman Realty and Construction Co., Inc., of New York, is a limit-height structure with a structural frame, including intermediate beams, weighing only 11 lb psf of floor area.

The light-weight construction of this building was made possible by changes in the Los Angeles Building Code, permitting the use of prefabricated light-gage steel floor panels, acting as transverse diaphragms to transmit lateral or earthquake forces. The structural frame is designed to resist vertical loading only.

Another weight-saving factor of this new building is the use of a false ceiling of vermiculite plaster, suspended 14 in. below the bottom of the floor beams. This fireproof envelope replaces the heavy encasement of each beam, which was required under earlier Codes. It reduces the dead load, cuts material costs, and saves erection time.

Each of the three 100 ft x 150 ft units has a 20 ft x 75 ft central core, providing elevator and air-conditioning services. The cores and some exterior panels have reinforced concrete walls 12 in. thick. Resistance to shear, designed into these walls, combats earthquake forces. The structural design gives complete freedom in placement of partitions. Under-floor ducts facilitate the installation of utilities.

Both the fabrication and erection of the 3500-ton steel framework for this unusual, glass-fronted structure were handled by Bethlehem Pacific Coast Steel Corporation, Bethlehem's subsidiary on the Pacific Coast.

BETHLEHEM STEEL COMPANY, BETHLEHEM, PA.

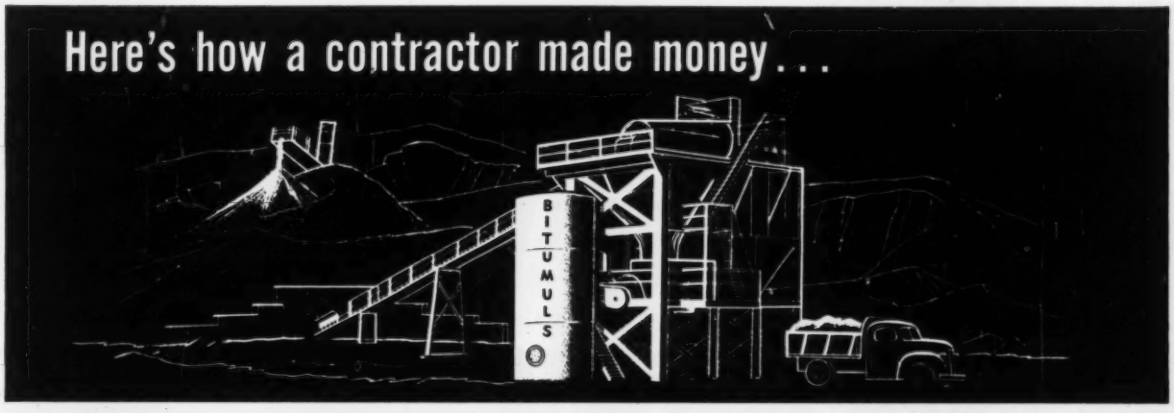
On the Pacific Coast Bethlehem products are sold by Bethlehem Pacific Coast Steel Corporation
Export Distributor: Bethlehem Steel Export Corporation

BETHLEHEM STEEL

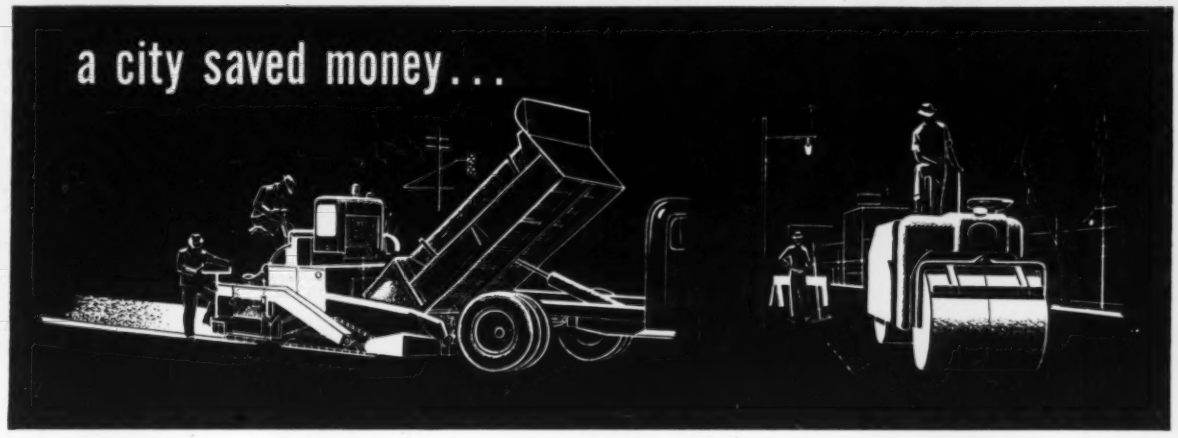
FABRICATED STEEL CONSTRUCTION

B I T U M U L S I S V E R S A T I L E

Here's how a contractor made money...



a city saved money...



...by bringing low-cost native aggregates up to "paving" standards with versatile BITUMULS®

IN EVERY LOCALITY there are low-cost aggregates—decomposed granite, red rock, bank-run gravel, shell, etc. Untreated, these materials are rarely suitable for base construction. When cold-mixed with small amounts of Bitumuls, however, they become stable and easily pass specification requirements for this type of work.

Municipalities get lowest paving costs when *maximum* use is made of these native aggregates and local plant mixing equipment. Typical are conditions in Oakland, California, where a local contractor conveyor-feeds waste bank-run gravel directly into hoppers that feed the material into his pug-mill. There it is mixed with a small quantity of Bitumuls and dropped directly into trucks that haul it to the job. This material, mixed cold and with damp aggregate, is ready for immediate compaction as base material on streets throughout the city. Costs are low due to use of marginal aggregate and utilization of spare time of the mix plant. *This results in profits to the contractor and lower costs to the city for high bearing base material.*

The wearing course may be asphaltic concrete from the same plant but with approved aggregate, or macadams bound with quick-setting grades of Bitumuls.

Throughout the country, Bitumuls is readily available from strategically located plants for on-job delivery.

There are Bitumuls Engineers in your area who know aggregate sources. They welcome an opportunity to consult you about your paving needs.

AMERICAN Bitumuls & Asphalt COMPANY

200 BUSH STREET • SAN FRANCISCO 4, CALIFORNIA

E. Providence 14, R. I. Perth Amboy, N. J. Baltimore 3, Md. Mobile, Ala.
Columbus 15, Ohio Tucson, Ariz. Seattle, Wash. Baton Rouge 2, La. St. Louis 17, Mo.
Inglewood, Calif. Oakland 1, Calif. Portland 7, Ore. Washington 6, D. C. San Juan 23, P. R.

There's always a job for a motor grader. This No. 12 maintained haul roads.

High-speed project at Travis Air Force Base

BIG YELLOW MACHINES USED BY MORRISON-KNUDSEN TO BUILD NESTING AREA FOR GIANT PLANES

Covering a 2,800-acre plain of the Sacramento Valley near Fairfield and Suisun, Calif., Travis Air Force Base handles some of the very largest bombers and transport aircraft. Training missions are flown regularly from there, and a large percentage of Korean casualties are winged from Japan to this base, which has the largest Air Force hospital on the West Coast.

When primary expansion at Travis called for larger plane parking areas, a high-speed contract was awarded to Morrison-Knudsen Company, Inc., Boise, Idaho. Additional construction included water facilities and an underground electrical system.

Plenty of heft—a D8 pulls a 100-ton Porter compactor, bearing down on sub-base fill for new plane parking area.

Every project has its problems, and this was no exception. To handle mammoth aircraft, runways and parking areas at Travis require thick asphalt and concrete surfaces. Sub-base material of deep rock layers increases their strength. However, there was far from an adequate supply of rock near the aprons for the new construction. This necessitated an endless chain of hauling units carrying fill rock from the stockpile to excavations, and returning with excavated earth.

Another problem posed was the fast production of aggregate material for concrete and asphalt surfacing. To solve this, a twin batching plant was erected at the base. Between them, the twin 4-compartment bins had a 320-ton capacity. Four sizes of concrete aggregate and sand, shipped in by rail, were trucked to the plant.

HALF-MILLION CUBIC YARDS JOB

Keeping a huge job moving on the double day after day requires, among other things, the use of tough machines. For this reason, Morrison-

Cat DW10 Tractor with No. 10 Scraper steps right along with excavated material, averaging 5 trips per hour on a 3-mile haul at Travis Air Force Base.

Knudsen concentrated largely on its Caterpillar equipment. And the big yellow Diesel tractors, 'dozers, scrapers and motor graders came through with outstanding performance.

In a well-organized setup, Cat-built units were busy all over the base. DW10 Tractors with No. 10 Scrapers, going 1 foot to 3 feet to 4 feet below the existing pavement, excavated and hauled poor soil and adobe. With built-up sides, the scrapers averaged 11 cubic yards per load. They made 5 trips per hour on a 3-mile round trip, over roads maintained by No. 12 Motor Graders.

Duties assigned husky D8 Tractors with 'Dozers were varied and many. One D8 worked in a borrow pit cleaning up around the shovel that turned out 350 to 400 ten to eighteen cubic yard loads of select material per day. Other D8s fed conveyors in the twin batch plant and stockpiled sand and gravel. Others were used to spread sub-base material and, where needed, to serve as pushers. Still another D8 pulled a 100-ton Porter compactor.

Totaling all earth and rock handled during the excavating and fill placement, M-K's equipment moved more than half a million cubic yards. Said Project Manager R. F. McCune: "Caterpillar-built machines enabled us to maintain our speedy schedule."

STANDARDIZATION PAYS OFF

Many contractors have found that standardization on rugged yellow equipment is good business. Every machine is engineered and built to stay on the job steadily in conditions that slow down ordinary machines. Other advantages include interchangeability of parts for simplified maintenance, increased operator efficiency and on-the-spot service from the nearby Caterpillar Dealer.

All this adds up to big production at rock-bottom costs. For further facts, see your Caterpillar Dealer. His file of performance data on construction jobs all over the country is well worth seeing. He is available at your convenience for information and a demonstration.

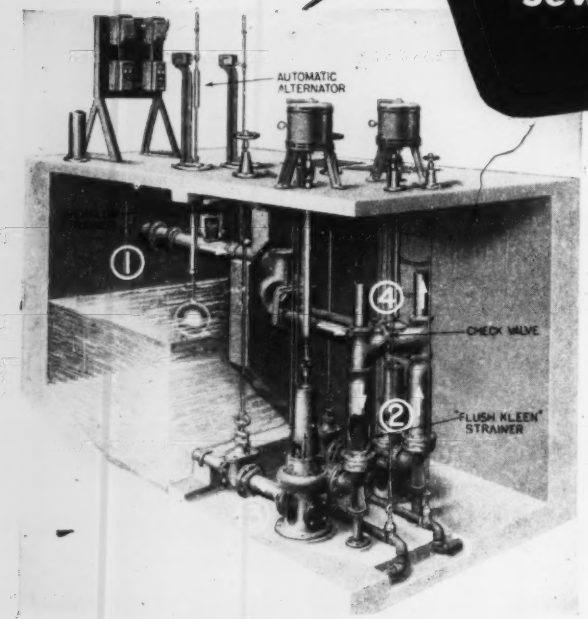
Two Caterpillar D17000 Engines power this 44-ton locomotive, used for switching cars at Travis Air Force Base.

Two D8s with No. 8S Bulldozers feed conveyors at twin batch plant for aggregate. D8s were also used to stockpile and spread sub-base fill.

CATERPILLAR TRACTOR CO., PEORIA, ILLINOIS

FLUSH-KLEEN
Sewage Ejectors

Over 12,000 In Service



Flush-Kleens are absolutely clog proof. Here are the reasons why: Flush-Kleen pumps automatically backwash the strainers, keeping solids from basin and pumps, with the impellers handling water only —this is accomplished as follows:

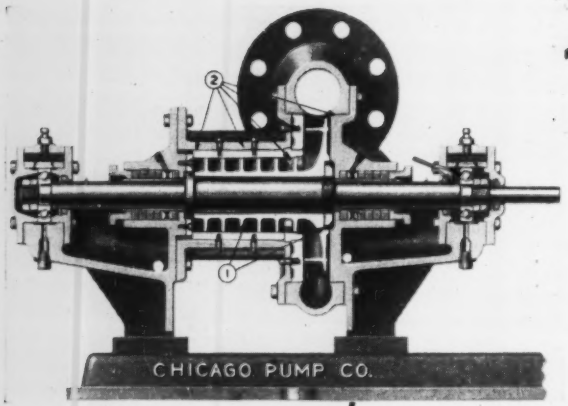
FILLING WET WELL . . . 1. Sewage flows through inlet pipe. 2. Coarse matter is retained on strainer. 3. Strained sewage flows through idle pump to wet well.

PUMPING . . . 3. Strained sewage is pumped from wet well. 2. Coarse matter is backwashed from strainer. 4. Special check valve closes; sewage and coarse matter are pumped to sewers.

CHICAGO SEWAGE EQUIPMENT

More Than 8,000 In Use

SCRU-PELLER
Sludge Pumps



Scru-Peller Pumps are simple in design, positive in operation and are truly clog-proof—here's why:

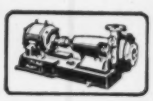
1. **SCREW AND IMPELLER** are keyed on the shaft. The screw has two flights and the impeller has two blades. Each flight in the conveyor connects directly with its own blade of the impeller.
2. **CUTTING EDGES.** There are four stellited cutting bars and a shear ring in the screw housing. Four more cutting bars are in the pump casing. The stellited edges of the screw and the edges of the impeller blades act against the cutting bars and shear ring, completely chopping all coarse solid material into small pieces that cannot clog or slow the pump.

CHICAGO PUMP COMPANY

SEWAGE EQUIPMENT DIVISION

622 DIVERSEY PARKWAY — **CHICAGO 14, ILLINOIS**

Flush Kleen, Scru-Peller, Plunger. Horizontal and Vertical Non-Clogs Water Seal Pumping Units, Samplers.



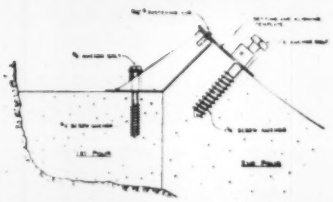
Swing Diffusers, Stationary Diffusers, Mechanical Aerators, Combination Aerator-Clarifiers, Comminutors.

Complete literature and engineering data will be sent on request.

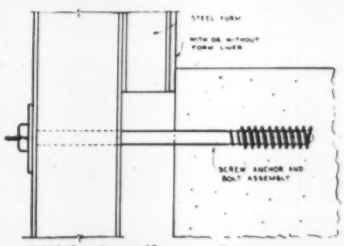
WHEN Heavy Anchorage IS REQUIRED . . . . .

SUPERIOR

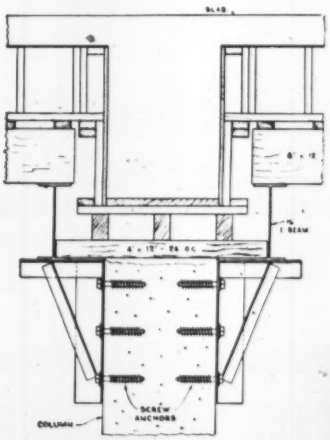
SCREW ANCHORS and BOLTS



For Anchoring Tunnel Forms



With Cantilever Forms on Dam Construction



Supporting Temporary Brackets for False Work.

PROVIDE TEMPORARY OR PERMANENT SUPPORT

Providing a tight safe fastening invulnerable to shock and vibration, Superior Screw Anchors and Bolts are widely used in heavy concrete construction for *temporary* anchorage for steel tunnel forms, false work support brackets, cantilever steel forms for gravity dams and similar structures. Their use also permits convenient lifting of precast concrete slabs, beams and piles.

Easily assembled, Superior Screw Anchors and Bolts are ideal for numerous practical applications such as *permanent* anchors for cleats, fenders and other accessories to concrete structures.

Anchor bolts can be removed quickly with a wrench. No strength is lost by returning the bolt to the anchor. The full bearing of the coarse bolt thread is on the firmly imbedded anchor which provides steel bearing for the thread.

Superior Screw Anchors are available for $\frac{3}{8}$" to $1\frac{1}{4}$" bolts.

Remember—when you use SUPERIOR you are assured of the *best* in design, material, and workmanship. Request a copy of our new Catalog 500—it contains a valuable table for spacing studs, wales, and form ties.

SUPERIOR CONCRETE ACCESSORIES, INC.

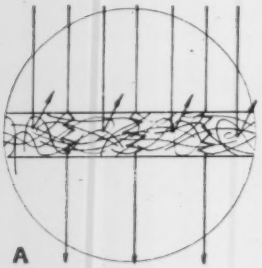
4110 Wrightwood Avenue, Chicago 39, Illinois

New York Office: 177 Broadway, New York 19, N. Y.

Pacific Coast Plant: 2100 Williams St., San Leandro, Calif.

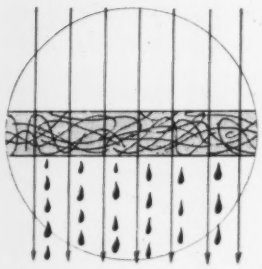
How tracing paper is made and why ALBANENE* is Different

TOUGH, LONG-FIBER PAPER
NOT TRANSPARENTIZED



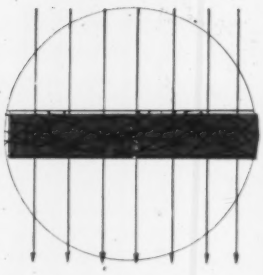
Diagrammatic enlargement of cross section of paper with high strength but low transparency. Fibers are surrounded by air, which has different index of refraction. Many light rays are bent back and do not get through.

SAME PAPER
TRANSPARENTIZED WITH FLUID MATERIAL



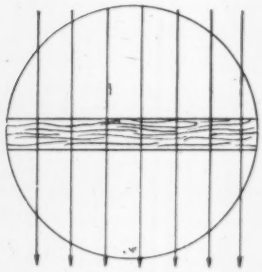
Same paper as "A", filled with oil or other fluid material, giving spaces between fibers same index of refraction as fibers. Reflection and refraction of light are reduced and paper becomes highly transparent. But transparency is not permanent because *fluids* "bleed" out.†

SAME PAPER
TRANSPARENTIZED THE ALBANENE WAY



Same paper as "A", filled with an inert synthetic resin, with correct index of refraction. This is how Albanene is made. Its transparentizer does not "bleed" out. Albanene holds its color and strength and is permanently transparent.†

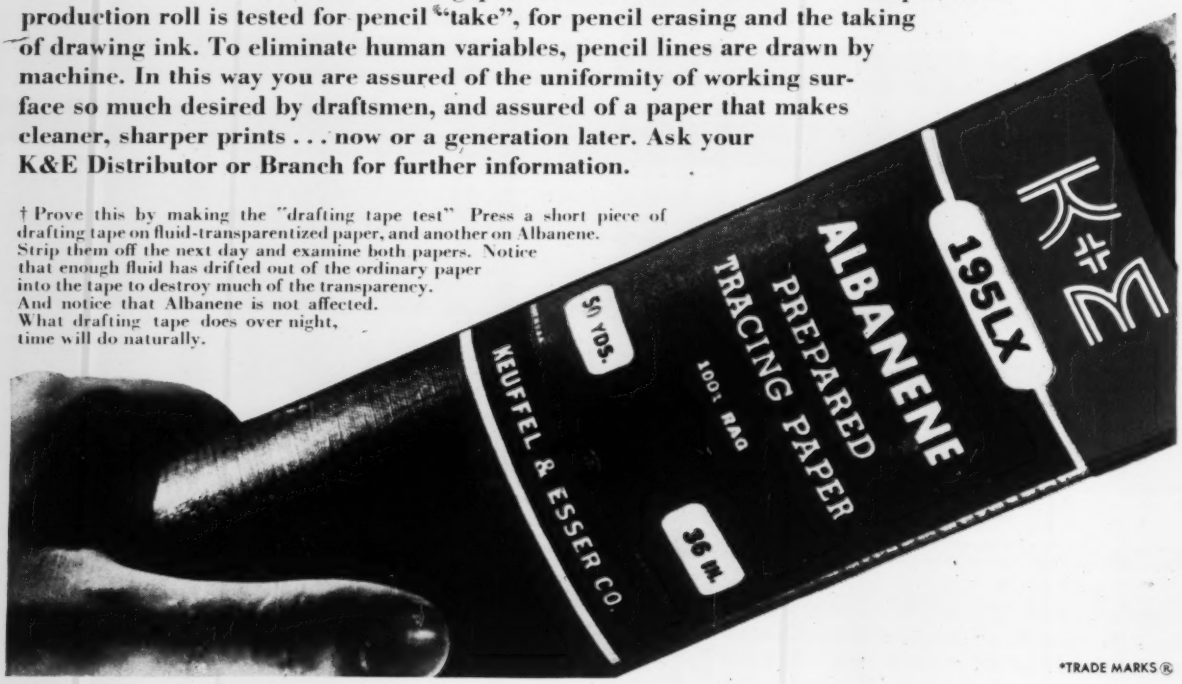
PAPER TRANSPARENTIZED BY
CRUSHING AND BEATING FIBERS



Papers are also transparentized at the mill by a "beating" process. The fibers are crushed, flattened and compacted. Reflection and refraction of light are reduced. But the process weakens the fibers and the strength of the transparent paper is low.

More than 15 tests are made during production of Albanene. For example, each production roll is tested for pencil "take", for pencil erasing and the taking of drawing ink. To eliminate human variables, pencil lines are drawn by machine. In this way you are assured of the uniformity of working surface so much desired by draftsmen, and assured of a paper that makes cleaner, sharper prints . . . now or a generation later. Ask your K&E Distributor or Branch for further information.

† Prove this by making the "drafting tape test" Press a short piece of drafting tape on fluid-transparentized paper, and another on Albanene. Strip them off the next day and examine both papers. Notice that enough fluid has drifted out of the ordinary paper into the tape to destroy much of the transparency. And notice that Albanene is not affected. What drafting tape does over night, time will do naturally.



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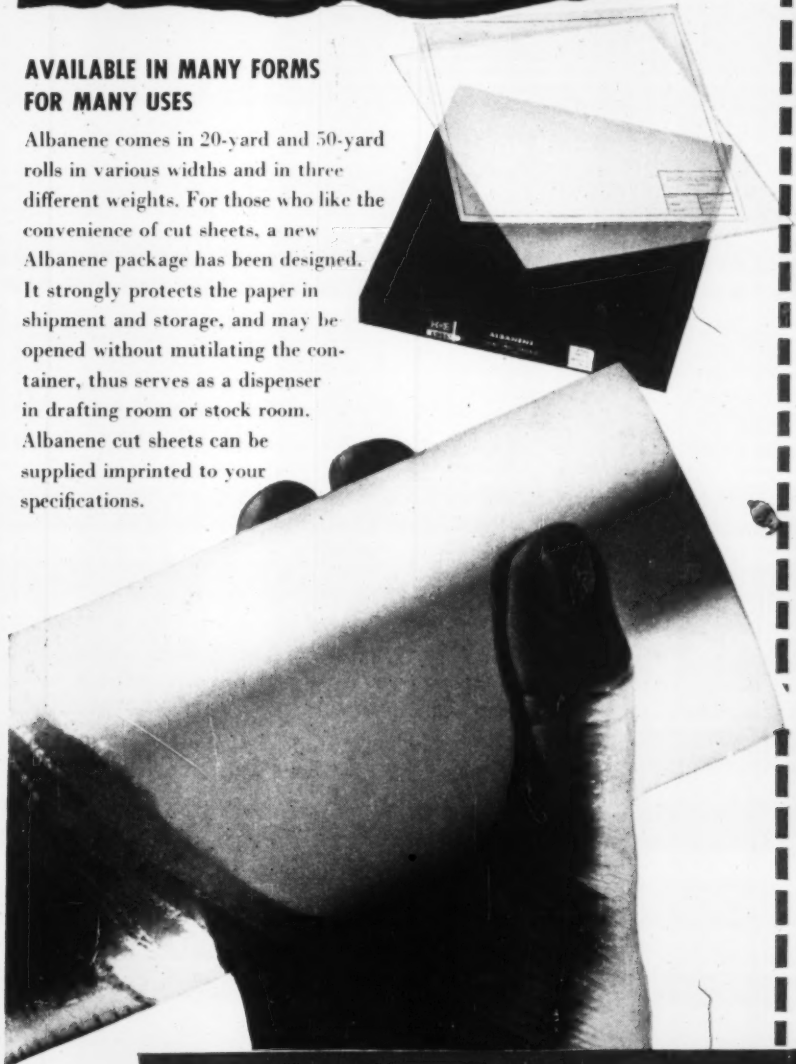
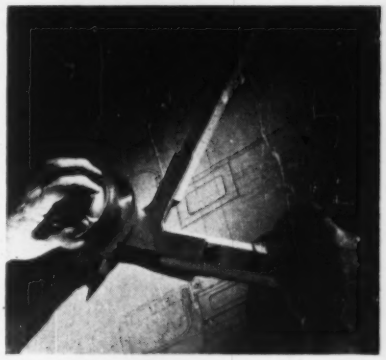


The Right Angle



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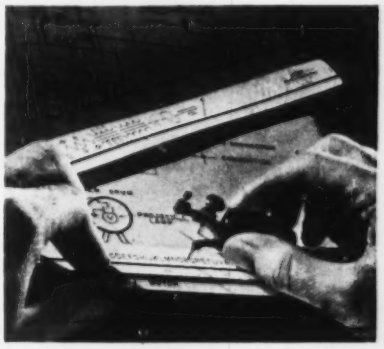

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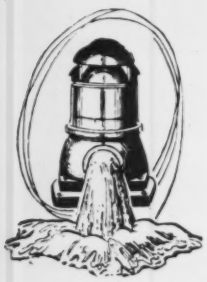

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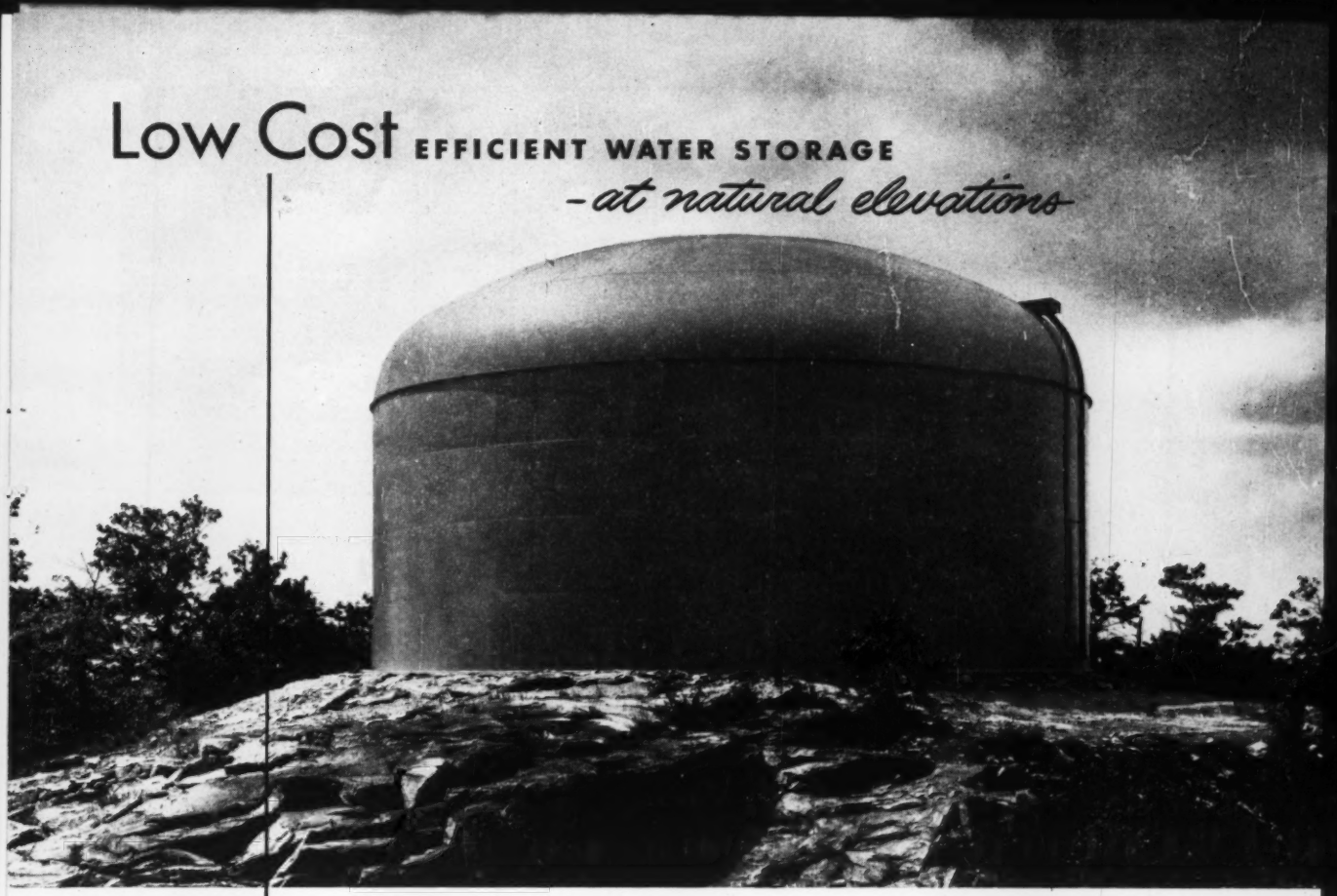
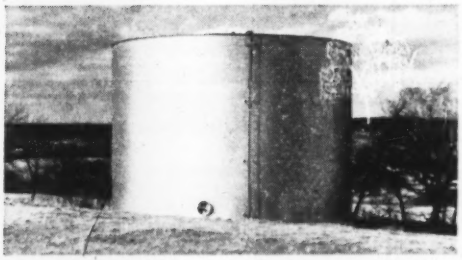
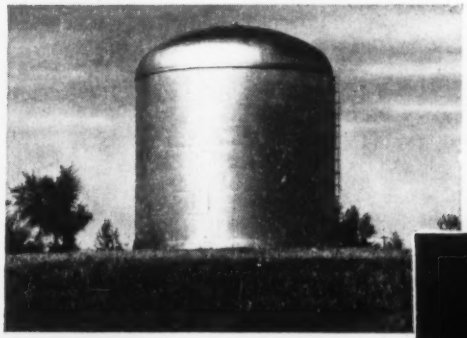


Worcester, Mass.—Ellipsoidal roof type, 1,500,000 gallons capacity.

* Steel Reservoirs



Liberty, N.Y.—Flat roof type, 300,000 gallons capacity.



St. Paul, Minn.—Ellipsoidal roof type, 2,000,000 gallons capacity.

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3. Windrow the salvaged material and load it out with scrapers, conveyor-type loaders, or other conventional equipment.
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5. Add small amount of binder to salvaged material (amount required varies with condition of binder remaining in salvaged aggregate).
6. Roll new surface (composed of salvaged material) using Hyster Grid Roller.
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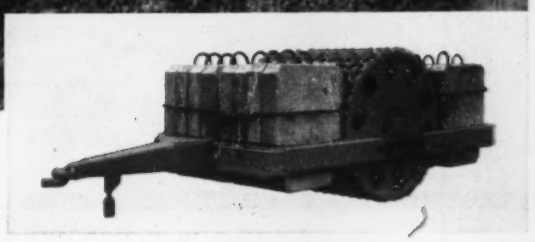

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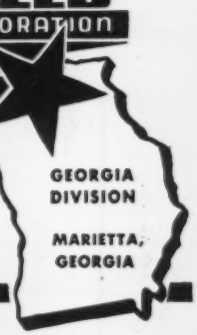

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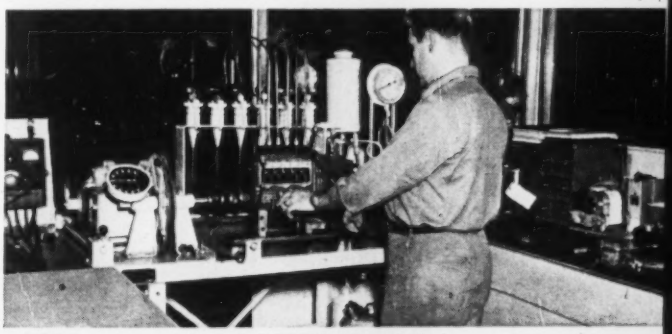

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CIVIL ENGINEERING

JANUARY 1953

THE MAGAZINE OF ENGINEERED CONSTRUCTION

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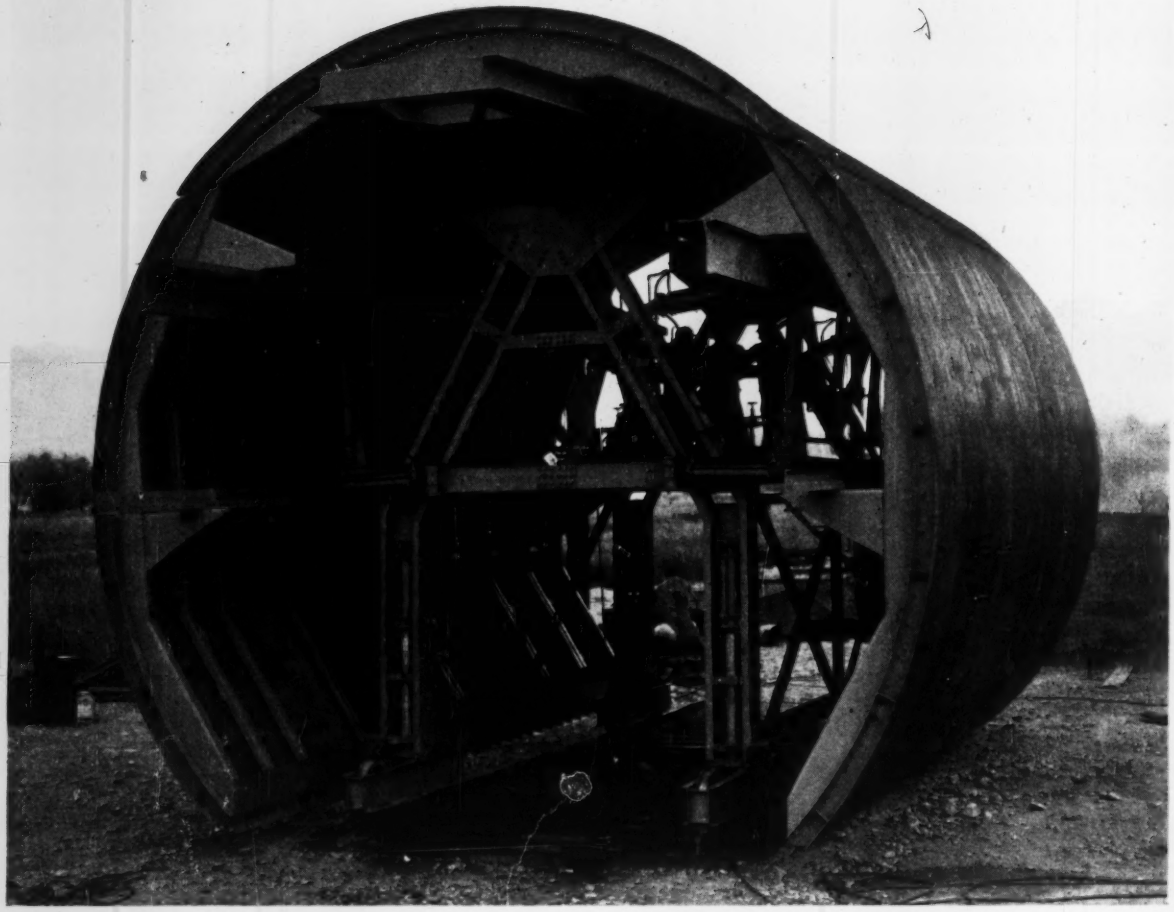

Horton Steel Form for Ross Dam Tunnel Linings

One hundred airline miles northeast of Seattle and a few miles south of the Canadian border, Seattle City Light's great Ross Dam is going up another 70 ft. Rising in staggered sections at the head of Skagit Gorge, Ross Dam is one of the most spectacular, difficult and challenging jobs ever tackled by engineers. Granite cliffs rise from the river at the dam site and every toehold for man or machine has been blasted from solid rock.

An important part of the project are the two tunnels that will carry water to the power house . . . and very important to the tunnels was the Horton steel form used to pour the linings. Horton steel forms are engineered to fit the job on which they are used. They are constructed for quick setting, accurate alignment, and easy stripping to assure maximum speed, important factors in the 2,000-foot long tunnels. Practical arrangement of hinges, rachets, jacks and other important details make for swift progress. Flush type access doors are specially built to assure tight fit and ease of operation.

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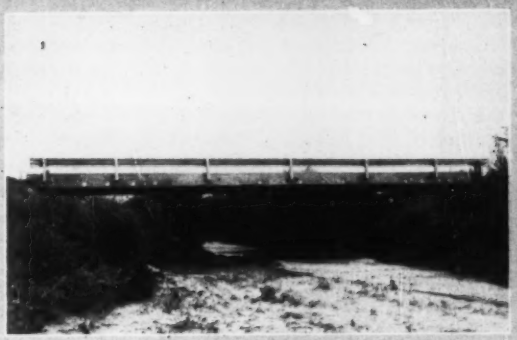
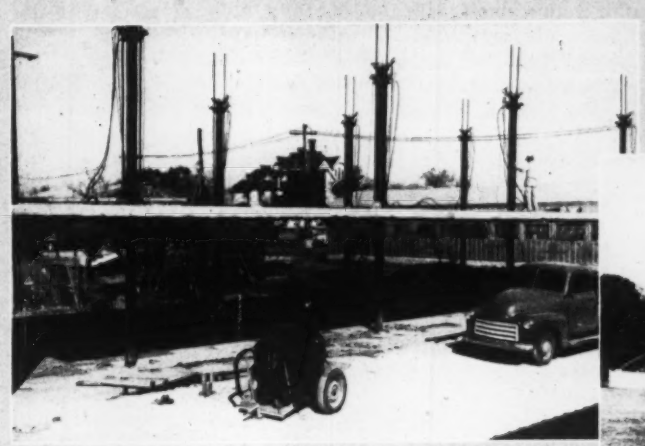
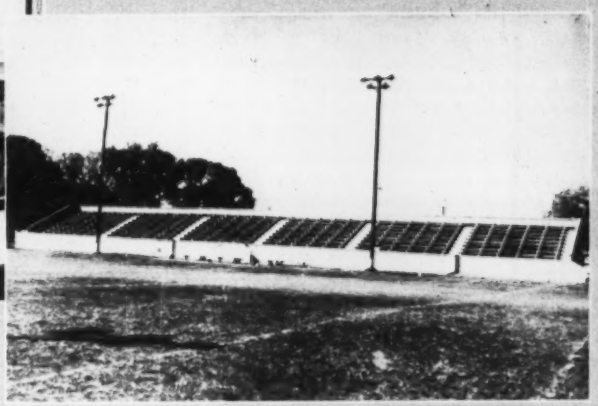
JANUARY 1953

THE MAGAZINE OF ENGINEERED CONSTRUCTION

At right: First prestressed bridge opened to traffic in U. S. was Turkey Creek Bridge in Madison County, Tenn. Three spans of 20, 30, and 20 ft were designed for H-10 loading, utilizing precast concrete-block elements with factory-tailored strands for prestressing. Below: On Southwest Research Institute laboratory building, in San Antonio, Tex., roof slab was cast at ground level, prestressed in two directions, and then jacked up to final position by Youtz-Slick method.







At right: First prestressed concrete block structure completed in United States is 1,500-seat stadium at Fayetteville, Tenn. Prestressed beams of 30-ft span are 12 in. deep.

Prestressed concrete makes rapid progress in the United States

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At the time of the first U.S. Conference on Prestressed Concrete (held at M.I.T. August 14–16, 1951) the use of circular prestressed concrete, including primarily pipes and tank structures, was firmly established throughout the world, and American firms were among the leaders in this type of construction. With respect to linear prestressed concrete, including beams, slabs and columns—the components which singly and in combination make up such structures as buildings and bridges—the situation was however very different. Virtually all the pioneering work in this field had been accomplished abroad. Over a period of at least ten years, our European friends produced not only a considerable amount of research but an impressive number of actual structures. In the United States, however, linear prestressing was a mere infant hardly a year old.

Only a few months before the conference, the first prestressed bridge

to be undertaken in this country was completed—the Walnut Lane Bridge in Philadelphia. Much of the interest in linear prestressing was fired by this bridge and in particular by the dramatically successful test of a full-scale girder similar to those incorporated in the actual structure. The imagination of many who gathered together at the first U.S. Conference on Prestressed Concrete had likewise been stirred by a new development—an American development—the combination of machine-made precast concrete-block elements with factory-tailored strand reinforcement units to produce prestressed beams and slabs. The first prestressed concrete bridge to be completed in this country, the Turkey Creek Bridge in Tennessee, was of such block construction.

There were, of course, other developments in linear prestressed concrete, some predating those to which I have referred, but the total American achievement, by any numerical measure, was very small in contrast to the intense interest evidenced by the conference participants in the summer of 1951. Probably less than ten actual structures embodying linear prestressing had been completed within the United States at that time.

Results of First U.S. Prestressing Conference

With what conclusions did the participants leave this first conference? Certainly the interest they had brought to the meeting was enhanced by what they had seen and heard. Many departed with a conviction that linear prestressed concrete would become an important addition to our structural resources. A very few, perhaps, were convinced that this material could never be adapted to our building economy. If any held this distinctly negative opinion, they were no doubt countered by a very few who were convinced that prestressed concrete would immediately assume a dominant role. The writer encountered no adherents of either of these unjustified points of view.

Rather, the sentiment of the majority seemed to be quite in keeping with the sound judgment which we like to think is typical of competent engineers. Enthusiastic about the potential importance of prestressed concrete, these men recognized that far more experience with actual construction would be needed before the ultimate competitive position of this material in the United States could be predicted. There were many basic questions which could only be answered by such experience, as well as others which would depend on future research and on the judgment of the men who would be charged with writing design specifications.

None of the questions that were raised implied uncertainty as to the structural excellence of the new material. They did reflect the dearth of experience in its application. Yes, the major need was for experience—experience for designers, contractors, and manufacturers. From such experience would come the vitally necessary cost data from which the economic merit of prestressed concrete relative to other forms of construction could be judged.

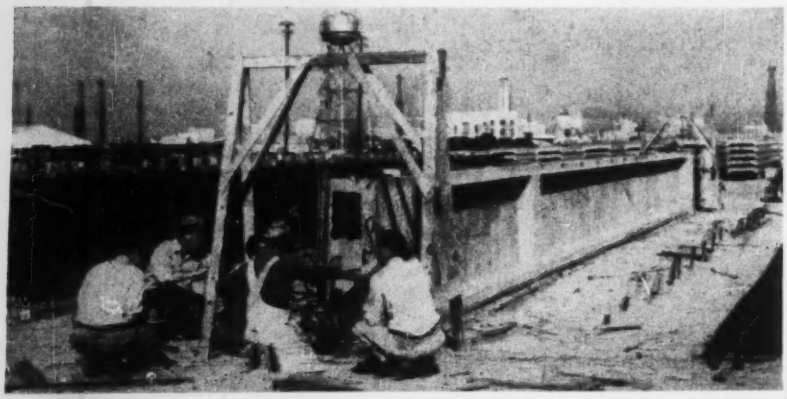
Cost data would be the primary yardstick by which new developments in prestressed concrete would be measured. Indeed, development and experience might almost be accepted as synonymous because development can be expected to be most intense during the early years of our experience. Finally, each new successful prestressed concrete structure might be expected to encourage other engineers to investigate the possible applicability of the material to their design problems. Thus experience would breed experience provided, of course, that each successive application reflected discrimination on the part of those responsible for the design and construction.

Specifically, what has been accomplished within the United States? This narrowing of the inquiry does not imply any lack of recent achievements in other lands. On the contrary, it must be acknowledged that they continue to lead us by a substantial margin. The long spans, the spectacular proportions, the indeterminate framing—all these are to be found outside of this country. It is to be noted, however, that such outstanding structures are now appearing in Latin America as well as in Europe. Although we have set no records for span lengths, for the application of prestressing to highly indeterminate framing, or in the production of structures of daring proportions, still it will be found that we have made progress. Quite properly our engineers wish to be sure that they can walk in this technique before they attempt to run. In this instance, to walk is not merely to gain confidence in the design and construction of prestressed concrete, but to establish the economy of the technique. It has been a wise course, calculated to assure the sound development of prestressed concrete.

In reviewing our progress, perhaps one should start with the most obvious evidence, that is, a recounting of actual construction. It is doubtful whether any individual is familiar with all the prestressed concrete structures completed, in progress, or under design. For the purposes of this report, the writer has utilized records supplied by the Portland Cement Association, has received help from several firms actively engaged in prestressed concrete design, and has, of course, scanned technical periodicals.

The picture, though certainly in-

Many applications of prestressing to building structures provide rather long spans, which might be provided by steel but which are beyond usual range of ordinary reinforced concrete. An impressive example of such an application is cotton warehouse at Long Beach, Calif., in which prestressed roof beams have span of 75 ft.



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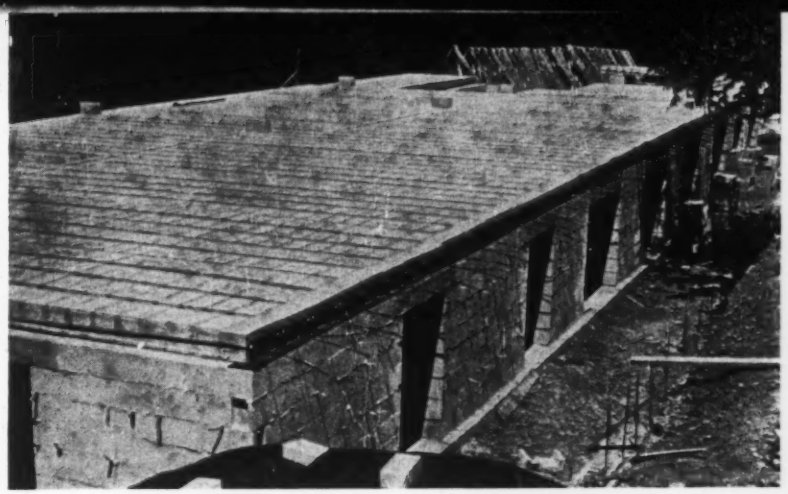

Floor of church auditorium in Nashville, Tenn., is formed of 8-in.-deep prestressed beams of 17 and 21-ft span made of precast blocks. To date in the United States, there have been no attempts to apply prestressed concrete to multi-story beam-and-column framing.

complete, is most encouraging. As of September 1952 there were in this country about 100 known projects incorporating linear prestressed concrete construction. Of these, about two-thirds are bridge structures, and about one-third building structures. There are a few which cannot properly be placed in either category.

Most of these 100 projects have either been completed or are under construction. With few exceptions, contracts have been awarded for the others. While supporting evidence is not available, it seems probable that prestressed concrete projects still in the planning stage would total at least another hundred. To appreciate the significance of the foregoing figures, we need only realize that they imply more than a tenfold increase in a period of one year.

Another interesting comparison is afforded by some statistics presented by Curzon Dobell, M. ASCE, at the conference last year. According to his estimate there were at that time about 175 bridges and 50 building frames in prestressed concrete outside of North America. Since the American effort was then negligible, Mr. Dobell's figures can be taken as the world total of linear prestressed concrete structures completed since introduction of the technique, or under construction up to the summer of 1951.

In terms of cold numbers, it is apparent that the American effort over a rather brief period has not been inconsiderable. It would be foolish to exaggerate the significance of these statistics, but surely they can be taken as evidence of intense and widespread interest in this new material of construction.

Variety of Techniques Employed

With regard to the total group of linear prestressed projects, perhaps one of the most interesting observations relates to the variety of prestressing systems employed. It will be recalled that the techniques developed in Europe were believed by many to be too labor-consuming to be competitive in the United States. However, the Freyssinet system, a European development, has been employed successfully in several projects in this country. The Lee-McCall system, which was developed and widely used in England, has been specified for certain recent structures. The high-strength bars used in the Lee-McCall system are soon to be produced in the United States by a new firm, the Stressteel Corporation.

While the foregoing foreign-developed systems have been widely used, there has also been widespread application of methods developed in this country. The "headed-wire" reinforcement developed by the Prestressed Concrete Corporation represents one of the important American developments. Of equal significance has been the development of factory strand reinforcement, which comes to the job in exact length and with end fittings attached. This system, pioneered by the John A. Roebling's Sons Company, and more recently available from other wire manufacturers as well, provides reinforcing units in a wide range of strand sizes.

All the reinforcing systems just described are employed for post-tensioned, prestressed concrete. Pre-tension reinforcement has been very popular for short-span, shop-fabricated construction. For these applications high-strength strand, of $^{5}/_{16}$-in. diameter or less, has been widely used. This is in contrast to the use of individual small-diameter wires typical of similar construction in Europe.

Before leaving the subject of reinforcement systems, it should be noted that, in addition to the types mentioned, others are known to be under development. Moreover, recently developed European systems will be applied to American construction in the near future. In the foregoing discussion, only those types have been included that are known to have been applied to actual construction in the United States.

Of the 65 or more prestressed bridge structures that have been undertaken to date in the United States, none is of record proportions, but two at least involve very large quantities of prestressed concrete. The Tampa Bay Bridge, designed by Parsons, Brinckerhoff, Hall and Macdonald, includes 363 prestressed concrete stringer spans, each 48 ft long. The prestressed construction in this crossing is noteworthy not because it involves more than three miles of structures but because the reinforcement will consist of high-strength, Lee-McCall bars, which had not previously been employed in this country.

The Garrison Dam Spillway Bridge includes 32 spans at 48 ft each. The Army Engineer Corps, for whom the bridge was designed, considered the use of prestressed concrete so appropriate on this project that bids were not taken on other forms of construction.

Prestressed bridges undertaken to date have been of several types. For spans up to 50 ft, pre-tensioned precast slabs have seen considerable use. As produced by the Concrete Products Company of America, these slabs are of rectangular cross-section, and are 3 ft wide. The depth depends on the span. Paper tubes are cast in the slabs to reduce their dead weight.

Machine-made concrete masonry blocks have been assembled and post-tensioned with strand reinforcement to form a beam. These beams, placed side by side, tied together with transverse post-tensioned steel, and topped with a cast-in-place concrete slab, have been used to form bridge decks with spans up to about 50 ft. In the system developed by Bryan and Dozier, the masonry blocks have a modified box section

with the upper flange narrower than the lower flange. Tensioned reinforcement is located outside the block webs. The cast-in-place concrete extends down into the space between adjacent beams and bonds to the tensioned strands. In the Anjoh system, the blocks are of I-section and the cast-in-place concrete does not extend between the webs of adjacent beams. In both systems, the compressive strength of the cast-in-place slab contributes to the strength of the deck.

Post-tensioned beams have been cast in their final positions in some instances, while in many other cases they have been precast and then moved into final position. It is interesting to note that while beams often have been precast at the bridge site, they likewise have been manufactured in central plants and transported several miles to the site. The latter method has proved practical for beam lengths as great as 65 ft when a plant equipped to produce the beams economically has been available.

Post-tensioned, monolithic beams have been of various cross-sections, but the I and T shapes have predominated. In many designs the beams, spaced a few feet apart, have supported a cast-in-place deck slab, the beams and slab acting as a composite section under live load. In other designs the beam top flanges have served as deck, the cast-in-place deck concrete being limited to the small strips between adjacent top flanges. In most designs the separate precast bridge beams have been provided with either precast or cast-in-place diaphragms, and the several beams have been tied together by transverse prestressed reinforcement.

Where monolithic post-tensioned beams have been employed, the tensioned reinforcement has usually been grouted in—even when it consisted of galvanized strand. This practice reflects a belief that grouting increases the ultimate strength of the member. Within the past year the U. S. Bureau of Public Roads has issued a "Design Criteria" for prestressed concrete bridges (post-tensioned), under which the grouting in of reinforcement is required.

Much of the foregoing comment on prestressed concrete bridge structures would apply without modification to prestressed concrete applications to buildings. In the latter field, too, there has been considerable variety. Both the older systems of reinforcement developed in Europe, and American innovations have been employed with success. Some designs have utilized machine-made concrete masonry units; others have incorporated single-cast prestressed girders, either precast or cast-in-place. In some cases prestressed beams have provided direct support for roof(or floor) decking. In other designs the decking has been carried by prestressed beams which were themselves supported by prestressed girders. The deck has sometimes been a cast-in-place slab, sometimes precast planks or slabs.

It is important to note that many of the applications of prestressing to building structures have provided rather long spans—spans that could be provided by steel girders but would be beyond the usual range of ordinary reinforced concrete. As an outstanding example of long-span application there might be cited the 94-ft roof beams which will span the gymnasium of the Bishop Du Bourg School in St. Louis.

Another impressive example is the 84 roof beams of 75-ft span which are incorporated in a cotton warehouse at Long Beach, Calif. Many other structures with spans almost as long as these could be cited, and of course there have been applications involving more moderate spans. It is apparent, however, that prestressed concrete has been very attractive in situations where rather long clear spans are essential.

The applications of prestressed concrete to buildings have included only a very few cases in which continuity over interior supports was utilized. This is in contrast to recent European practice in which continuous spans have been executed with considerable ingenuity. Until similar construction is attempted within the United States, we cannot say whether continuity will provide savings in material which more than offset possible increases in labor costs. There have been no attempts to apply prestressed concrete to multi-story beam-and-column framing. However, even outside the United States, applications of that kind are few in number and of rather recent origin.

Any outline of achievements of the past year in the field of building structures would be incomplete without reference to the prestressed concrete roof slab incorporated in a laboratory building at the Southwest Research Institute, San Antonio, Tex. This flat slab, without columns or drop panels, is 88 ft by 38 ft by 6 in. thick, and is supported by steel columns spaced 24 ft apart in each direction.

The slab was cast at ground level, prestressed in two directions, and then lifted to its final position by the Youtz-Slick method. The engineers responsible for this undertaking concluded that prestressing techniques will permit considerable increases in slab spans beyond those now practical with ordinary reinforced concrete. They further concluded that whereas shear stresses at the column zones are of critical importance in ordinary flat slabs, these stresses are of minor significance in two-way-prestressed slabs.

Progress Toward a General Specification

The question of specifications merits some comment. At the closing session of the conference last year, the participants unanimously adopted a resolution urging the American Concrete Institute to prepare, as soon as possible, a proposed standard for nomenclature and design in prestressed concrete. At the time of the conference there already was an ACI committee which numbered among its objectives the preparation of such standards. A few months thereafter this committee was expanded into the Joint ASCE-ACI Committee on Prestressed Concrete. This joint committee has now assembled a proposed nomenclature and is working toward a proposed specification for the design of prestressed concrete.

Opinion among those who are active in the field of prestressed concrete is sharply divided on this subject of a general design specification. Some believe any such specification would inevitably be restrictive and would stifle development. This group prefers to place reliance on the good judgment of those skilled engineers who are pioneering in the development of the new technique. A second group believes that we have progressed beyond the period in which designs are performed by a relatively few specialists. They assert that the number of engineers engaged in the design of prestressed concrete is large and is growing rapidly. They believe that "proposed" or

Walnut Lane Bridge in Philadelphia, completed early in 1951, was first prestressed bridge to be undertaken in this country. Much of the interest in linear prestressing in the United States was fired by this bridge.

"tentative" specifications would be an invaluable aid to these engineers. Finally, they feel that the danger that such specifications will retard development can be avoided by frequent reviews and revisions. While recognizing the merit in both of the two points of view, the writer believes that the arguments of the latter group will prevail.

The "Design Criteria" of the U. S. Bureau of Public Roads for post-tensioned, prestressed concrete bridges has already been mentioned. This rather brief document probably has influenced the design of some of the bridges built during the year, but the extent of its influence is unknown to me. No similar specification relating to other types of prestressed concrete structures has yet appeared in the United States.

Significance of Activity to Date

It is not sufficient merely to record the remarkable activity that has been so evident during the past year. In addition one must inquire as to what conclusions are to be deduced from this activity. There is ample proof that the enthusiasm so evident a year ago was not superficial. Rather it has spread among a much larger segment of the profession. A part of the experience so badly needed has been acquired during the months past.

What of economy? Consider first bridge construction. In many cases the decision to use prestressed concrete may have been influenced by a desire to try out the concept, despite probable higher costs. Such a decision, on the part of a public agency responsible for the construction of large numbers of bridges is entirely sound. Experience is gained thereby, actual cost data are obtained, and a contribution is made to the development of a method of construction that has great economic promise.

Certainly it should not be inferred that all the bridges built to date were built at a premium. Quite the contrary. There is evidence that many won in competition with non-prestressed, competitive types. Of course it is very difficult to assemble cost data for the many prestressed bridges that have been constructed across the country. Nevertheless we will soon have a most reliable indication of the competitive position of prestressed concrete. If those agencies which have already contracted for prestressed bridges contract for additional bridges of similar types, we may reasonably assume that such structures are competitive.

From the standpoint of economy, the activity in prestressed building construction is particularly encouraging. While competitive bids are not ordinarily available for alternate designs of a building, neither architect nor engineer will knowingly specify an uneconomical design. When prestressed elements are used in building construction, it may be assumed that such members provide the most economical solution to the specific problem. Some decisions may have been influenced by a temporary scarcity of steel, but even in these cases it is unlikely that prestressed concrete would be substituted were it not reasonably competitive.

In closing, a few conclusions or summarizations may be in order.

The number of prestressed concrete structures that have been undertaken during the past year is impressive. It is of interest to note that foreign developed techniques, as well as American innovations, have been used with success.

In many cases, prestressed concrete has been selected as the result of successful competition with unprestressed alternate designs. In a few instances the governing factor has been the shortage of steel. There have also been instances, particularly in the field of bridge construction, where prestressed concrete has been selected at a known premium in the interest of long-range benefits.

The prestressed bridges have been of short to moderate span, although there have been examples of multi-span construction involving very substantial contracts.

In the field of building construction there has been evidence of variety but the frequent use of rather long-span prestressed beams and girders has been noteworthy.

While American achievements in linear prestressed concrete still lag behind the accomplishments in other lands, the events of the past year justify a belief that the gap may be closed in a relatively few years. Certainly these events comprise a big step toward establishing the position of prestressed concrete as an important, competitive structural resource.

Indeed this has been a year of progress in prestressed concrete.

(The paper from which this article is taken was presented by Mr. Holley at a joint session of the ASCE Structural Division and the American Concrete Institute, under the chairmanship of A. E. Cummings of the ASCE Research Committee, at the Chicago Centennial Convention.)

Forging-hammer foundation built to control destructive vibrations

Spring-mounted inertia blocks of grout-intrusion, prestressed concrete prove value

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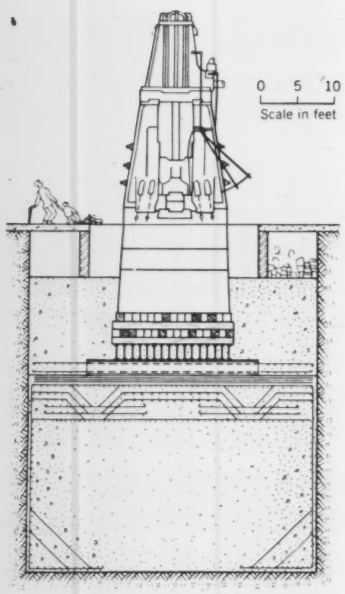


FIG. 1. In conventional foundation for 55,000-lb die forging hammer, anvil rests on timber pad over concrete block. This is a British setting for drop-hammer installation.

Forging hammers consist essentially of a large anvil mounted on a concrete foundation with a timber pad between the anvil and the foundation, as shown in Fig. 1. The pad protects the foundation from the enormous shock generated when the anvil is struck; it acts as a spring which obeys simple harmonic motion with the weight of the anvil on top of it. Not only the anvil but the foundation and the ground below act as a weight on the surrounding springy earth, thus forming a complex oscillatory system of two springs and masses, each weight having six degrees of freedom and giving twelve natural frequencies to the system. It was discovered before the war that the ground has a natural frequency of its own. A very soft clay may oscillate naturally at about 10 cycles per second, while a stiff dry sand may oscillate at upwards of 30 cycles per second.

As a result of the hammer blow, the whole system receives a wave of shock which travels through it and makes the whole system oscillate, each part at its own natural frequency. Should any two parts have the same or approximately the same natural frequency, they will be in resonance and will build up a destructive amplitude. It is considered essential to so arrange a hammer foundation that no destructive resonances will occur.

Hammer manufacturers have been trying since 1843 to solve this problem. Civil engineers have now succeeded in solving it through foundation control, by considering the machine, its foundation, and the surrounding ground as a single unit.

In designing the foundation for a new 8-ton forging hammer for the International Nickel Co. at Huntington, W. Va., the primary requirements of the owner were that previous enormous ground vibrations be eliminated and that, if possible, production costs be lowered by decreasing maintenance costs. In some cases hammer maintenance has been extremely high, absorbing up to 85 percent of the available working time. The foundation was to be designed for a hammer actuated by super-heated steam at higher-than-normal pressure, producing almost the greatest possible shock in general forging practice.

This 8-ton hammer imparts a blow every three-quarters of a second; so

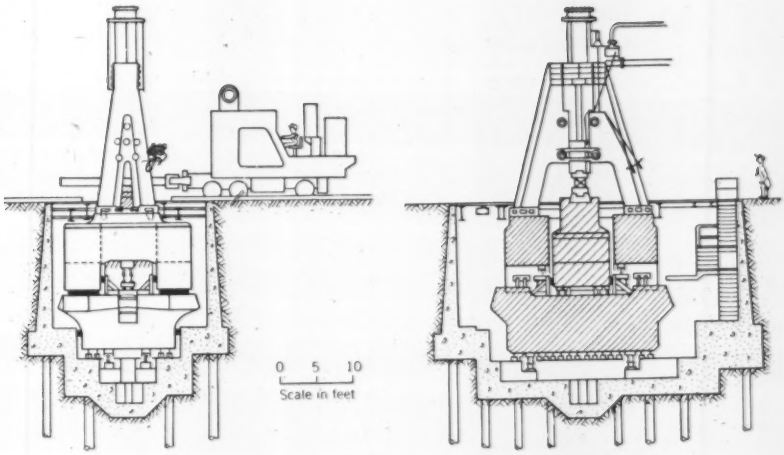


FIG. 2. In new-type setting for 8-ton steam-actuated forging hammer built for International Nickel Co., Huntington, W.Va., spring-mounted concrete blocks effectively control heavy and destructive shocks.

Foundations for forging hammers have long presented problems because of the tremendous vibrations which quickly destroy ordinary concrete. The new-type foundation here described employs heavy springs to cushion the steel anvil in every direction, as well as cushioned blocks of very high strength concrete to carry the 8-ton steam-actuated hammer and the steel anvil. This installation bids fair to improve production from nickel and nickel-alloy ingots by eliminating many costly maintenance delays due to fatigue failures in hammer and foundation. The article is based on the paper given by the authors before a joint session of the American Concrete Institute and the ASCE Structural Division, presided over by A. E. Cummings, chairman of the ASCE Research Committee, at the Centennial of Engineering Convention in Chicago.

all the resulting oscillations of anvil and foundation, together with hammer superstructure, must be completed and the entire installation brought to quiescence again within this period. In addition, the amplitude of the hammer movement must be small both to avoid discomfort to the operating crew and to avoid resulting decreases in production. These are the factors which control the sizes of the masses employed, the various constants of the springs used and their position, the amount of damping, and the positions of the dampers between the several masses.

In this case, illustrated in Fig. 2, the superstructure was given an additional block of anular form surrounding, but free of, the anvil. Both this block and the anvil are mounted on independent rubber springs which sit on a lower concrete block, which in turn is mounted on helical springs resting near the bottom of the reinforced concrete pit, so as to reduce the live load on the soil to a few tons. A sump provided at the bottom of the pit catches condensate water and any ground seepage. The oscillatory system is illustrated schematically in Fig. 3. It will be realized that the anvil mass, and the superstructure mass on the upper block, have an exact oscillatory relation to each other via the lower block.

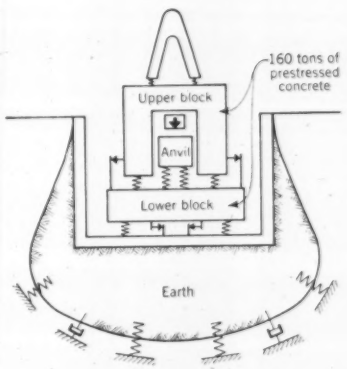


FIG. 3. Schematic diagram explains action of oscillatory shock-absorbing system for 8-ton forging hammer shown in Fig. 2 and described in this article.

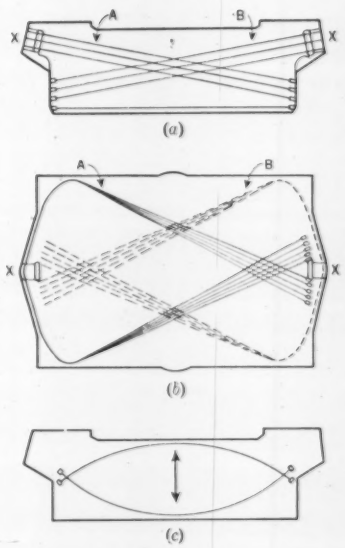
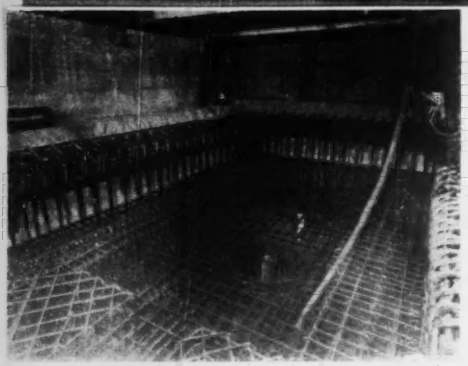
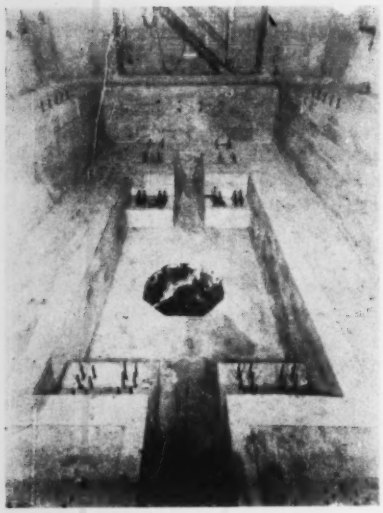


FIG. 4. Lower concrete block was prestressed in three dimensions by jacking from ends of block only, as shown here schematically. Freyssinet flat jacks were placed at points *X* in (*a*) and (*b*).



As foundation for "floating" forging hammer installation, existing concrete-lined pit was deepened, and in it heavy reinforcement was arranged for floor and collar, which were placed by conventional methods. Construction rests on existing foundation piles.



Completed floor of pit is ready for mounting of springs and dampers that will support lower inertia block.

Design Problems Solved

The importance of this installation is that it is the first forging-hammer foundation to be fully analyzed for vibration, movements and loads in all directions; the first to be designed to reduce hammer maintenance; the first to be designed to prevent or minimize fatigue failure in itself; the first to be built of prestressed concrete; and the first in which specially designed high-strength grouted concrete is used.

All parts are completely visible and accessible for maintenance, a point considered to be of paramount importance in hammer foundations. If any part fails it can be removed without disturbing the rest of the installation. With conventional foundations the hammer nearly always has to be dismantled and the anvil jacked out of the foundation before any repairs can be made—a long and costly operation, during which production ceases.

It is interesting to compare this installation with the conventionally mounted 55,000-lb drop hammer illustrated in Fig. 1. The live load transmitted to the ground by this hammer is upwards of 2,000 tons for each blow, producing a true earthquake shock, noticeable up to two miles away. The superstructure sits on top of the anvil and is thereby subjected to its tremendous shock, although the foundation is much simpler with this arrangement, even if a vibration-controlled type of foundation is used.

The length of the shock wave which travels through the anvil and the rest of the system is only a few feet long when very hard nickel alloys are hammered. It is this very short wave length that creates special problems. The shock wave reaching the bottom of the anvil is partly reflected back again, but a small amount is refracted through into the rubber spring, and then from the rubber through into the concrete. Shock waves in compression, when reflected from a concrete-air boundary, are reflected in tension. Should these waves come to a focus at any point, the tension will build up sufficiently to cause failure. The concrete blocks therefore must be given a special shape so as to cause the stress waves inside them to disperse rather than come to a focus.

High-Strength Concrete Needed

It is known that concrete can fail in fatigue just as does steel, especially when the water-cement ratio is high and the concrete new. After a few hundred thousand reversals of stress, the strength of concrete is reduced to perhaps half its original value, particularly when the stress is oscillated from tension to compression and back again. The disrupting effect of strongly oscillating forces is very considerable; even in the fully controlled setting here described the live load from the anvil is about five times its dead load.

For these reasons it was decided to use a very-high-strength concrete with a low water-cement ratio and to prestress it in all three directions simultaneously, although this had not been previously done anywhere.

In order not to disturb the production lined in the plant, an existing concrete-lined pit was selected as the site for the new foundation. The pit was

Sequence of construction of

1. Formwork is constructed and Freyssinet anchor cones are placed, as well as Korfund helical spring housings, and some prestressing cables and reinforcement.
2. With aid of temporary top formwork struts, vertical steel is positioned and pipes and tubes are tack welded to auxiliary reinforcing bars. In this view, temporary struts have been removed following tack welding of vertical steel. With drain pipes, Lee-McCall holding-bolt tubes, and Prepakt intrusion tubes also in position, aggregate is placed to depth of 30 in.
3. Remaining cables and reinforcing steel are installed and aggregate is placed to

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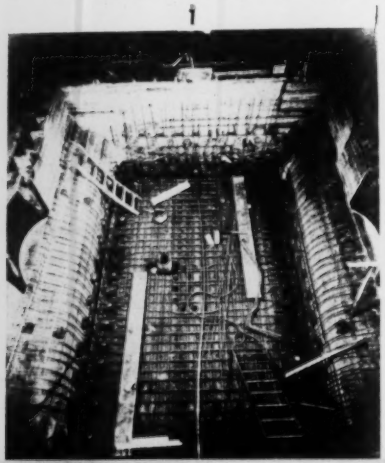

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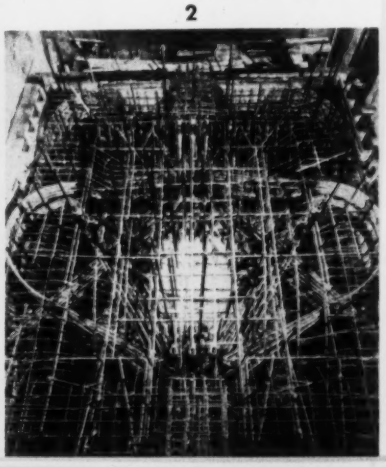

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deepened and a new bottom placed on top of existing foundation piles. At the floor level of the old pit a heavy reinforced concrete frame was formed and placed to take both the inward thrust of the earth and the outward thrust to be imposed by the side springs from the lower block. The walls of the pit were left in place.

The pit was so narrow it was impossible to obtain both the width required for the lower inertia block and the 3-ft working space around it required for the stressing jacks. Because of limited room for jacking, a method of arranging the stressing cables was developed whereby the whole of the three-dimensional stressing could be arranged by jacking from the two ends of the lower block only. This was done by making the cables start at one end of the block, pass diagonally through to the other end, turn there through fairly large radii, and then return to the same end at which they started. The diagrams of the "A" and "B" cables in Fig. 4 (*a*) and (*b*) show how this was done and also how the block was compressed vertically at the same time by making these cables slope up from opposite sides. In addition, the cables shown in Fig. 4 (*c*) were included to compress the whole center of the block vertically. The considerable amount of friction between the prestressing cables "A" and "B" and the sheaths in which they slide was overcome by inserting two flat Freyssinet jacks at the points marked "X" in Fig. 4 (*a*) and (*b*). The jacks were inserted with a saddle on top of them so that, when they were inflated, the cables would be drawn out and pulled slightly around the difficult corners.

Although Freyssinet cables were used to take up all calculated bending moments, mild steel bars were placed just under the entire surface of both blocks to prevent the formation of hair cracks. In dynamic structures the stress concentrations around cracks lead to fatigue failures which spread throughout the whole member.

To prevent possible slippage of the Freyssinet cables through the Freyssinet anchor cones under the extremely high dynamic stresses, repeated millions of times in a year, the cable strands were threaded through holes drilled in a $^3/_8$-in.-thick mild steel plate placed outside the cone after the stressing operation was completed. The plate, along with the cable strands themselves, were later grouted in solid, as shown in Fig. 5.

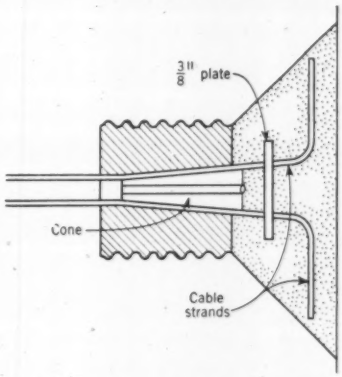


FIG. 5. In Freyssinet anchorage for ends of prestressing cables, cone is wedged securely in center of spread strands, individual strands fitting into grooves around cone. Strands then are threaded through holes in $^3/_8$-in. plate, bent over, and grouted into solid block of concrete.

Upper Block Also Mounted on Springs

The upper block weighs almost the same as the lower block—160 tons. It is mounted on four massive precast legs, so as to provide access space to side-controlling springs and dampers, and to the sides of the anvil and its springs. The very topmost part of an anvil can become red hot after prolonged hot forging. Since the rubber springs under the anvil must on no account be allowed to rise very much in temperature, provision was made for cooling them by water spray whenever the bottom of the anvil reached a temperature of 120 deg F.

All the rubber springs which act sideways are mounted on welded steel brackets and these brackets are held down tightly to the lower block by high-tensile-steel bolts using the Lee-McCall system of stressing bars developed recently for prestressed concrete. These bolts were jacked at their lower ends to a stress of 30 tons per sq in., while the brackets were suitably shimmed, so that the springs have a pre-compression such that under no conditions of oscillatory movement will they ever become loose or chatter, even though there may be a very large eccentric blow causing considerable sideways and rotational movement of all masses. The largest eccentric blow occurs when a 2-ton weight, lifted by the overhead traveling crane, is swinging 40 ft onto the top side of the anvil for driving in the keys, or fixing wedges, used for holding the anvil pallet tightly in place.

It is to be noted that the hammer superstructure and the anvil are precisely placed in relation to each other and that there is no risk of their get-

100-cu yd lower inertia block:

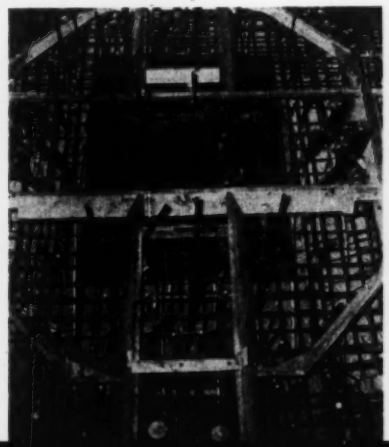
within 12 in. of top of form.

4. Top formwork and base plates are positioned and remaining aggregate placed. Pressure form is installed over top, except over base plates.

5. High-strength grout is pumped into aggregate through intrusion pipes extending to bottom of forms.

6. After pressure top and rest of forms have been removed, steel bearing plate for anvil can be clearly seen in middle of block, and in corners, bearing plates for four legs to support upper block. At age of six months, 6 × 12-in. test cylinders showed strength of 10,800 psi.

4



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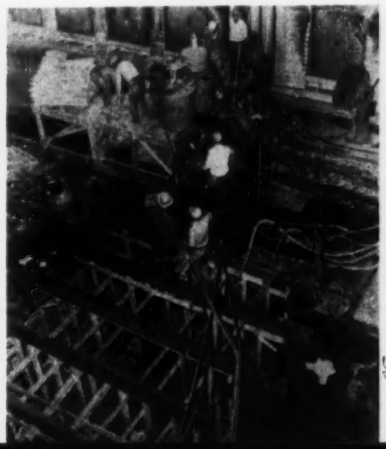

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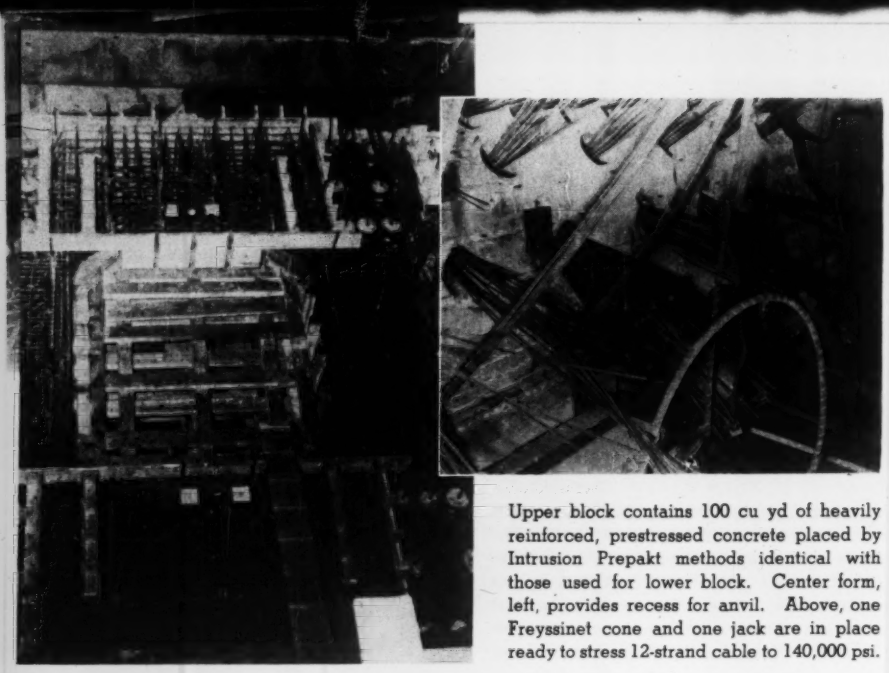
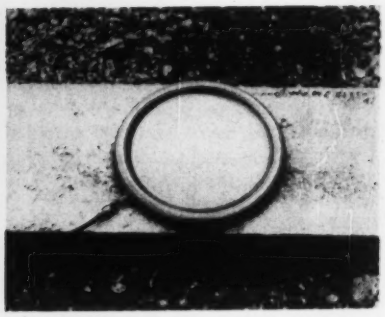


Upper block contains 100 cu yd of heavily reinforced, prestressed concrete placed by Intrusion Prepakt methods identical with those used for lower block. Center form, left, provides recess for anvil. Above, one Freyssinet cone and one jack are in place ready to stress 12-strand cable to 140,000 psi.



Freyssinet flat jacks, above, placed in pairs at middle of prestressing cables, when inflated, helped to tension long cables as they passed around bends in both upper and lower blocks. This procedure in lower block is illustrated in Fig. 4, where X's indicate location of flat jacks.

ting out of position once they have been properly adjusted and the rubber springs have taken up their initial and working creep.

The hammer was next mounted onto the upper block, with a 1-in.-thick high-quality timber pad between, this pad being grouted with cement and sand to make it fit the top surface of the block and the under side of the hammer base plate perfectly. Instead of the old $3^1/_2$-in.-dia steel holding bolts, high tensile steel bolts of $1^1/_8$-in.-dia were used to hold the hammer in place, but were not stressed, to avoid risk of cracking the base plate.

Complicated Construction Problems Solved

Although this foundation is of concrete, a common building material, its construction is not in the same class as an ordinary foundation. The construction of the pit bottom was not unusual except for the large amount and close spacing of the reinforcing steel, shown in a photograph. The building of the two inertia blocks, each having a mass of approximately 100 cu yd, was however extremely complicated.

The main factors contributing to the difficulty of the operations were: (1) the requirement that the concrete have the highest possible degree of homogeneity, (2) the complexity of the stressing cables and reinforcing steel, (3) the very restricted working space available for almost every operation; and (4) the necessity for extreme accuracy in every operation, almost approaching that prevailing in a machine shop. Actually the two inertia blocks should be thought of as machine parts, although they are part of the hammer substructure, or foundation.

The 135 cu yd of concrete for the replacement of the pit bottom were required to have a strength of 4,200 psi as measured in 6 × 12-in. cylinders. The concrete used in the pit bottom consisted of Type 2 portland cement, 7.5 sacks per cu yd, crushed limestone sand with a fineness modulus of 2.79, and No. 4 to $^3/_4$-in. crushed limestone with a fineness modulus of 6.70. The mix used was 1:2.01:2.35 by weight, with a slump of from 3 to 4 in. The average strength of 6 × 12-in. cylinders made from this mixture was 4,837 psi at 28 days.

Concrete for the upper and lower inertia blocks, because these blocks are parts of a dynamic foundation, was required to be far better than that used for ordinary high-quality construction. The prime requisite was that this concrete be absolutely homogeneous, in so far as this term can be applied to concrete, in order not to disintegrate during the passage of stress waves. Secondly, it was stipulated that the ultimate strength be in order of 8,500 psi on the basis of 6 × 12-in. cylinders. Third, because this concrete was to be prestressed by post-tensioning, a strength of approximately 4,000 psi was required at 7 days. In addition to these rigid requirements, the complexity and close spacing of the reinforcing steel and stressing cables necessitated extraordinary placeability. A high cement content could not be used because of the danger of excessive drying shrinkage.

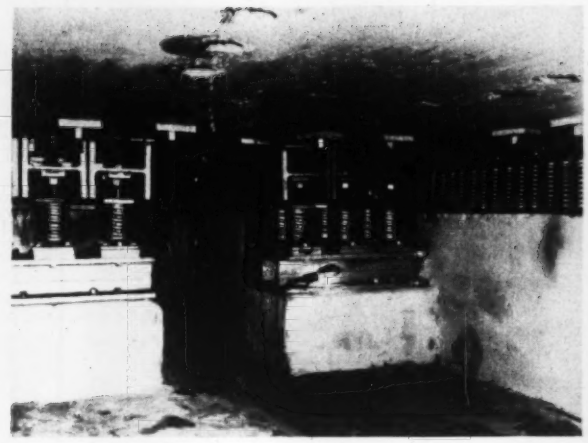
It developed further that the five steel base plates on the lower block would have to be set and aligned before any concreting was started. These base plates consist of a large one in the center for the anvil and a smaller one at each corner to carry the four legs of the upper block. These five base plates cover 25 percent of the area of the top of the lower block and are so located as to prevent workmen from having access to the lower part of the block.

The presence of these base plates made it practically impossible to obtain a homogeneous mass of high-strength concrete if conventional methods of placement were followed. Study determined that placement by the Intrusion-Prepakt method offered the greatest promise of a homogeneous mass provided that the required strength could be obtained.

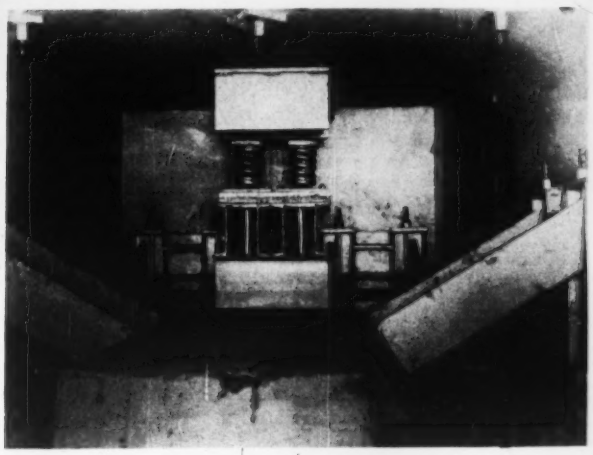
The Prepakt method consists of first packing the form with dry or damp coarse aggregate and then filling the voids through intrusion tubes, $^3/_4$ in. in diameter, with a grout mixture. The grout is a mixture of portland cement, finely divided pozzolanic material designated by the trade-name Alfesil, sand, an intrusion aid, and water. Several functions are claimed for the intrusion aid, namely, that as a protective colloid it inhibits early stiffening of the grout and therefore acts as a fluidifier, that it tends to hold the solid constituents in suspension, and that it neutralizes the effects of drying shrinkage.

Mix Developed by Research

As Prepakt concrete with the high strength desired for this installation had not been previously made, a considerable amount of research work was necessary to develop the mixture



Left: Lower block rests on series of helical springs around its perimeter. (Those on one side appear at extreme right in photo.) Spring adjustable friction dampers, one under each corner of block, are seen at center and left. All are accessible for adjustment or replacement.



to be used. A series of preliminary tests were made using Type 2 portland cement, crushed limestone sand, Alfesil and intrusion aid in the grout, with crushed limestone as coarse aggregate.

After extensive tests it was decided to use grout made up of a 9:1:6 mixture of cement, Alfesil and sand. Tests of Prepakt concrete made with this grout resulted in a compressive strength of 4,070 psi at 7 days, 5,850 psi at 28 days, and 7,250 psi at 90 days. The cement content was 7.75 sacks per cu yd of concrete.

Compression tests of 6 × 12-in. cylinders of Prepakt concrete with the 9:1:6 mixture used in the blocks, gave strengths of 4,010 psi at 6 days, 5,705 psi at 27 days, 7,020 psi at 60 days, 8,990 psi at 90 days, and 10,600 psi at 180 days. Further tests will be conducted after one year. The increases in strengths from 28 days to later periods are somewhat higher than would normally be expected.

This high-strength grout also was used to grout the stressing wires in their sheaths. Here it was necessary to inject a slow-setting pumpable grout through an orifice approximately $^1/_4$ in. in diameter and then through the stressing cable sheaths for distances of as much as 65 ft. In addition to the other characteristics given, the 9:1:6 grout has a final setting time of 38 hours, and an expansion after 3 hours of 11.6 percent by volume.

The replacement of the pit bottom, consisting of approximately 135 cu yd of concrete, was carried out as a normal construction job. No particular difficulties were encountered in this work other than those of placing the concrete around the very closely spaced reinforcement steel and finding room for the workmen to vibrate it properly.

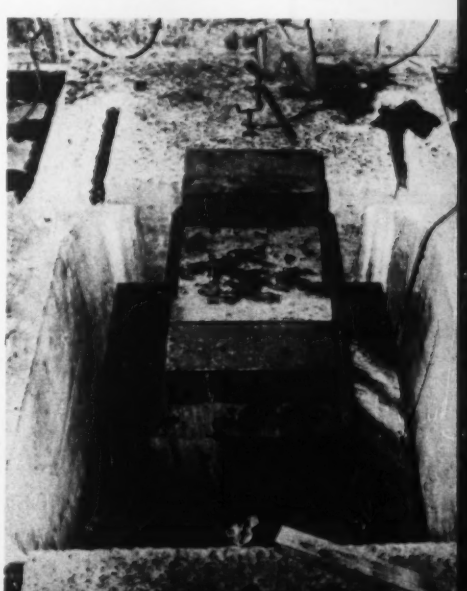
Before work was started on the lower inertia block a scale model was built. The cost of building it was quite high but the model gave those concerned with the installation a much clearer picture of the finished work than was possible from a study of the drawings. It gave the steel workers an opportunity to plan the sequence of operations, thus saving a considerable amount of time in the actual installation, and it made it possible to foresee any mistakes that might occur, again saving time in the actual installation.

The pressure top on the lower block forms was stipulated by Intrusion-Prepakt Co. It consists of a layer of muslin which is set against the concrete, backed up by a layer of fine wire mesh and a layer of expanded metal lath, the whole backed up by and attached to wooden batten strips, fabricated with spaces approximately $^1/_2$ in. wide between the boards. The purpose of this top is to allow air and any surplus water to escape as the grout rises in the form, and to provide a smooth surface strong enough to stand the pressure needed for filling all corners and voids with grout.

The Freyssinet stressing cables are

Above: Access space between upper and lower blocks allows full inspection, adjustment or replacement of springs supporting upper block and anvil. Welded steel brackets seen at each side of this photo mount rubber springs to prevent side motion. Brackets are held down by prestressed Lee-McCall stressing bars and nuts.

Below: Steel anvil is seen seated in recess in upper block. Anvil rests on rubber springs on lower block.



made up of twelve wires 0.20 in. in diameter, evenly spaced around a central wire spring and encased in a thin metal sheath. The wires used have a carbon content of 0.70 percent, a yield stress of approximately 150,000 psi, and a tensile strength of approximately 210,000 psi.

The thin metal sheathing for the prestressing cables furnished by the Freyssinet Co. was so brittle that cracks developed where the cables were bent sharply. In view of this, standard steel pipe of 1-in. dia was used for sheathing the sharply curved cables in the upper block. Although the actual installation of the cables with the pipe sheathing was somewhat more difficult than with the thin metal sheath, the extra work was offset by the smaller amount of inspection required to prevent grout leakage.

On the afternoon before grouting the lower block, approximately $5^1/_2$ tons of ice were spread over the top surface and sprinkled with water. The chilled water seeped down through the aggregate and reduced its temperature to approximately 50 deg F by the next morning.

Grouting was started at about 8:00 a.m. but, because of some mechanical trouble with the pumping equipment, was not completed until about 1:00 a.m. the next morning. Under normal conditions the grouting could have been completed in about ten hours.

To minimize further the temperature rise of the concrete, cooling water was pumped through the drain pipes, located in the central part of the block, throughout the entire grouting operation and for some time thereafter. Within three hours after grouting was completed, water sprays were directed at the outer surfaces of the block, and these surfaces were kept in a saturated condition for several days.

The pressure top bracing was removed the second day after grouting was completed, and all forms were removed the fourth day. Removing the forms was somewhat more difficult than in conventional concrete work because of the penetrating nature of the grout, which seizes at every hole or crevice.

Building and grouting the precast legs for the upper block and the upper block itself followed the same general procedures that have been described for the lower block.

Tensioning the cables was started nine days after grouting was completed, at which time the concrete had attained a compressive strength in excess of 4,000 psi.

The specifications for prestressing stipulated a stressing load of 140,000 psi, and in so far as we have been able to determine, this stress has been put into all the cables, although actual stress measurements could not be made on some because of the curvature to which they are bent.

After stressing, the cables were grouted with a Prepakt grout similar to that used in the blocks. Metal plates were then placed over the wires and the wires bent preparatory for secondary concreting, as shown in Fig. 5.

Good Installation Behavior

Before the hammer was placed in operation, vibrograph readings were taken of all main movements of anvil and upper and lower blocks, to check frequencies and amplitudes under different working conditions. The various frequencies of all masses have properly missed the natural frequency of the ground, previously measured at 8.5 cycles per second.

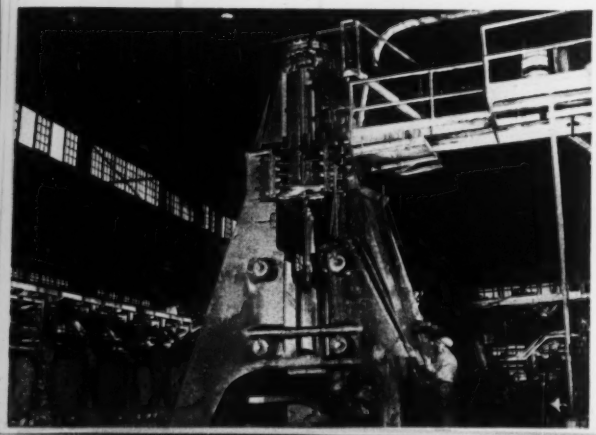
This installation was completed in October 1951, and the hammer placed in operation on the 29th of that month. Numerous inspections of the concrete have been made since that time and with one exception the only defects observed have been cracks in the secondary concrete. This cracking, which is important only as regards appearance, is undoubtedly due to lack of sufficient bond between the secondary and primary concrete.

The only unsatisfactory Prepakt concrete found in the installation is directly under the anvil base plate. In January 1952, a little more than two months after the hammer began work, the rubber springs under the anvil appeared to have shifted. Inspection early in February indicated that the steel base plate under the anvil was sinking slightly into the concrete. When the anvil had been lifted and the rubber springs and base plate removed to permit an examination of the concrete itself, it was found that the depression in the concrete varied from nothing to slightly over $^1/_2$ in. The surface of the concrete otherwise was found to be firm, hard and satisfactory.

Apparently, during the grouting operation the froth that seems to be inherent in Prepakt concrete was not completely forced out from under the base plate. It appears that this froth was not of uniform thickness and under the impact loads imposed during hammer operation quickly broke down and was then washed away by the very rapid movement of the cooling water every time a blow was struck.

To correct this condition the anvil was jacked up, without dismantling the hammer, and the existing concrete chipped out to a depth of approximately 6 in., without disturbing the reinforcing steel, after which the chipped surface was sand blasted. Using additional reinforcing steel, a low platform $5^1/_4$ in. high of 6,400-psi concrete was placed to support the base plate. Since August 1952, when the hammer was put back in operation, there has been no indication of failure of this repair work.

The authors wish to acknowledge the assistance of D. B. O'Neill, London, in connection with general design problems; L. P. Price, Paris and London, in connection with stressed concrete; A. Talbot, London, advanced vibration mathematics; Raymond E. Davis, M. ASCE, and assistants, grouted concrete research; L. O. Walcutt, International Nickel Co., general supervision; and Niels Thorsen, Freyssinet Co., supervision of stressing operations. General construction of the installation was by the International Nickel Co.; and grouting was done by the Prepakt Concrete Co., Cleveland, Ohio. The rubber springs were supplied by the Andre Rubber Co., Ltd., London, and the helical springs and friction dampers by the Korfund Co., New York. The installation was designed by J. H. A. Crockett, London, A. M., Institute of Civil Engineers; and technical supervision of construction was by A. M. Klein of the Robert W. Hunt Co., New York, N.Y.



Completed 8-ton steam-driven forging hammer seen in process of finishing 4 × 11-in. blooms from ingots. Blooms are used for rolling sheet bar on 24-in. mill.

Endicott Avenue Bridge over Boston & Maine Railroad crosses at an angle of 68° 40′. Bridge consists of precast, prestressed concrete beams of T-section placed so that inverted T's touch at bottom. In this view all beams are in position ready for transverse prestressing.

J. C. RUNDLETT
Bridge Engineer
Massachusetts Department
of Public Works
Boston, Mass.

Short-span prestressed beams carry highway bridge in Danvers, Mass.

Our introduction to prestressed concrete in the Massachusetts Department of Public Works will seem elementary and a very timid approach to a means of construction that has been proved by others for a period of years. Since we in Massachusetts are conservative by nature and trusting only to a degree, our introduction to prestressing was akin to a visit to the dentist. We knew that we should make this visit, but whether it would be painless or not, we did not know.

An opportunity to make the acquaintance of prestressing arose early in the summer of 1951, when we were considering the replacement of the old single-span wooden-stringer skew bridge carrying Endicott Street over the single track of the Boston & Maine Railroad in Danvers, Mass. At the time there was a great deal of uncertainty about the availability of steel for bridge construction because of the critical situation in Korea. With this rather dismal outlook for the future, it seemed that we should look into prestressing not only to realize an immediate saving of steel but also to make ourselves thoroughly familiar with this type of design and construction.

When the under-clearance requirement for the proposed bridge was raised by over 2 ft, it seemed imperative to keep the superstructure depth to a minimum. Studies for depth indicated that a concrete beam-and-slab superstructure would be about 2 ft 8 in. deep, a steel stringer with spiral reinforcement about 22 in. deep, and a concrete slab about 18 in. deep. As the bridge was over a railroad, the desirability of a structure requiring no falsework and a minimum amount of forms, pointed to a precast, prestressed inverted T-section with the bottom flanges butted, and with a total depth of 17 in. (Fig. 1).

Cost Estimates

The cost of either a steel-stringer or a concrete beam-and-slab bridge was estimated to be about $58,000. To keep the cost of the prestressed bridge from exceeding that amount, we felt it necessary to get a bid of about $150 each for the beams complete in place. This seemed at the time a reasonable price for a beam containing only $^3/_4$ cu yd of concrete. Knowing that unfamiliarity with the subject on the part of the contractors would tend to increase the cost, we decided to go ahead with this design and pay the additional cost if it did not prove to be exorbitant.

The induction of prestressing forces is made, in the case of a post-tensioned beam, by applying a permanent load at the ends of the beam at such points and in such a manner as will give initial compressive stresses in the concrete in the bottom of the beam. In this case it was felt that the simplest system for applying these forces would be through wire strands and end fittings of the Roebling type, cast in the beam but not bonded to it. These strands are simple to install and can easily be stressed or restressed.

Ordinarily, on a short span such as this, pre-tensioning would be used, but local firms had no casting beds suitable for pre-tensioning. Then, too, we were looking ahead to the building of longer-span girders, for which post-tensioning is superior, and felt that the Danvers job would give us invaluable experience in preparation for the building of such larger structures.

Provision is made on the deck of the bridge for a 40-ft roadway and two 6-ft sidewalks. The final span is 27 ft $4^1/_2$ in. center to center of bearings, with an overall length of 29

ft. The angle of skew is 68 deg 40 min. The 47 beams are dimensioned as shown in Figs. 1,2 and 3.

The structure was designed for an H-20 loading, in accordance with the specifications of the American Association of State Highway Officials, so that the beams and the cast-in-place slab would be composite under live load.

Prestressing Specifications

The design specifications stipulated that the strands used were to be galvanized and unbonded; that the initial stress in these strands would be 120,000 lb; and that the residual stress, assuming a 15 percent loss, would be 102,000 lb. To gain this stress, the initial tensioning of the strands would be 13 tons. The initial allowable compressive stress in the bottom of the concrete was to be 2,000 psi. The cracking load of the beam was to be the dead load plus twice the live load, including impact, and the ultimate load was to be not less than the dead load plus $3\frac{1}{2}$ times the live load. Concrete with a strength of 3,750 psi was to be used in the cast-in-place portion.

The specifications called for 5,000-psi concrete with a minimum cement content, per cubic yard of concrete, of 860 lb. The average weight of the fine aggregate was to be 1,220 lb, and that of the coarse aggregate, 1,670 lb. All concrete was to be air-entrained, and the maximum slump allowed was 3 in.

The wire for the strands was to have the following properties: an ultimate strength of 220,000 psi minimum; a 0.7 percent elongation at 175,000 psi; a minimum ultimate elongation in a 10-in. gage length of 4 percent; and a wire diameter of 0.196 in.

Prestressing units were to be furnished complete with factory-attached terminals, and the strands were to be prestretched. Before being placed, the prestressing elements were to be given a light coating of

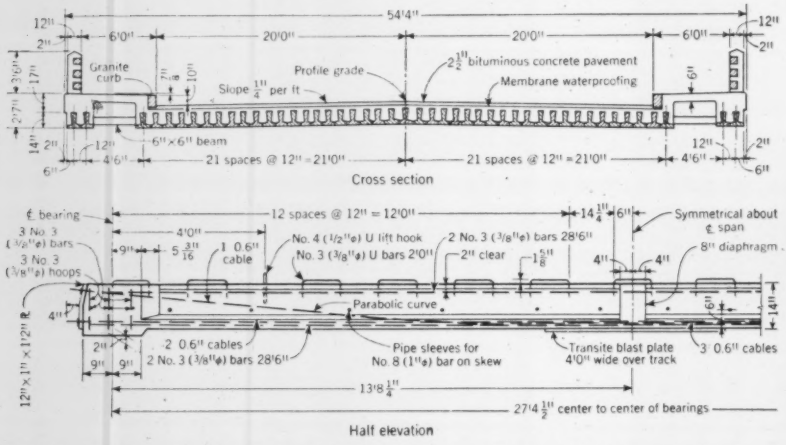


FIG. 1. Floor system for Endicott Street Bridge in Danvers, Mass., over Boston & Maine Railroad, consists of 47 precast beams, post-tensioned with Roebling wire cables.

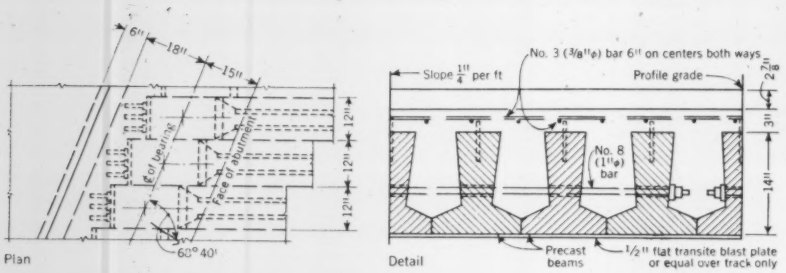


FIG. 2. Skew crossing and limited clearance indicated use of precast beams in combination with cast-in-place concrete floor slab.

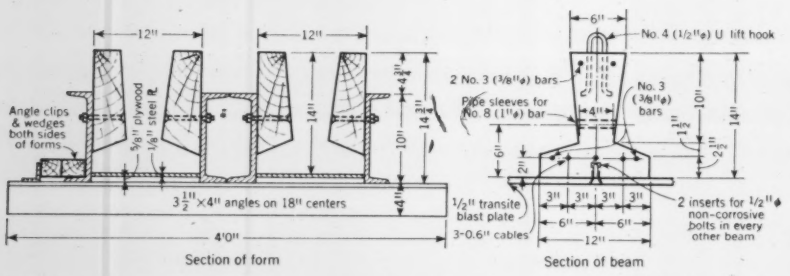
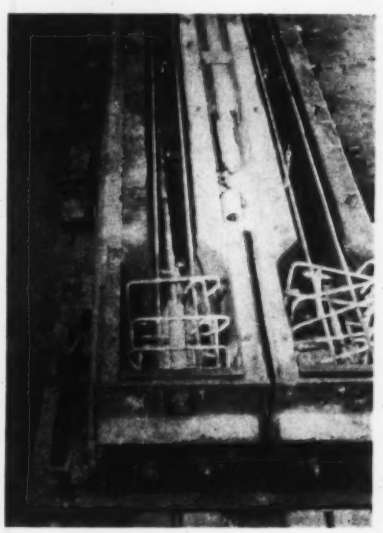
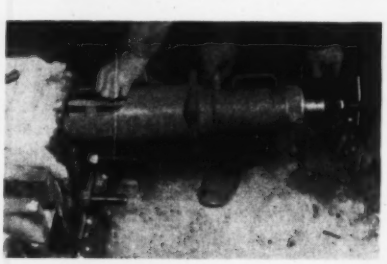

Angle clips & wedges both sides of forms
5/8" plywood
1/8" steel
3 1/2" x 4" angles on 18" centers
4'0"
Section of form
No. 4 (1/2"ϕ) U lift hook
2 No. 3 (3/8"ϕ) bars
Pipe sleeves for No. 8 (1"ϕ) bar
No. 3 (3/8"ϕ) bars
1/2" transite blast plate
3-0.6" cables
2 inserts for 1/2"ϕ non-corrosive bolts in every other beam
Section of beam

FIG. 3. Beam forms provided for casting of two beams at one time. Unbonded cables were tensioned after two days at 13 tons each.



Sequence of operations may be summarized as follows: Beams were cast, two at a time, in forms shown above, where wrapped strands and reinforcement are seen in place. After casting, beams were prestressed by jacks attached to ends of rods, as seen below. Minimum interval between pouring and stressing was about 2 days.



grease and wrapped with two layers of sisal-kraft paper to prevent any bond between the concrete and the strands. Before stressing, the concrete was to attain a strength of at least 3,300 psi.

On August 14, 1951, bids were opened for the furnishing and delivery to the site of the 47 beams in the bridge and for 3 test beams, to be constructed and stressed either in the shop or at the site. The New England Concrete Pipe Co. was the low bidder at a price of $164 per beam, or a total of $7,708 for the 47 beams; the Nelson Cement Stone Co. was second with a bid of $176 per beam; and the Berke-Moore Co. third, with a bid of $276.40 per beam.

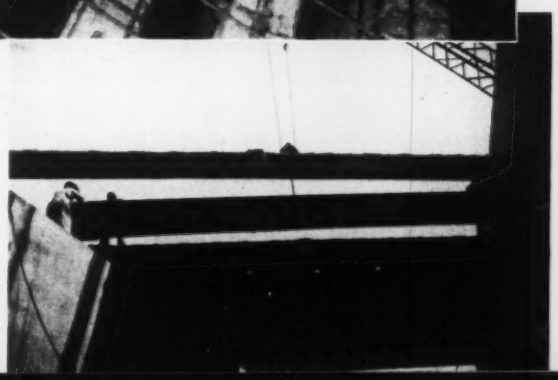
The contract for constructing the bridge, which included placing the beams, was awarded on October 30, 1951 to the low bidder, the Coleman Brothers Corp. Their price for the placing of the beams was $2,000. The price of the second bidder for placing the beams was $750, that of the third, $2,000, and that of the fourth, $2,380.60. Thus, the total low bid for the fabrication and delivery of the 47 beams amounted to $7,708, plus the $2,000 for placing the beams. A comparison of the cost of furnishing with the cost of placing the beams indicated that the bid of $2,000 for the latter was exorbitant. This observation was verified when all the beams were placed in a little over 8 hours at a cost to the contractor of about $350. The remainder of the general contract prices were considered satisfactory, so both contracts were awarded to the low bidders.

The New England Concrete Pipe Co. chose to fabricate the beams at its Providence, R.I., plant, which is about 65 miles from the bridge site. This contractor developed a form that would permit the casting of two beams at one time (Fig. 3). It stood up well for the entire pouring period. The base for the forms was made up of 4 × 3½-in. angles, 18 in. on centers, with a common center form made up of channels and milled lumber attached to these angles. The outside forms were held at the required location by wedging against angles welded to the bottom supporting angles, and the top of the outside form was held by metal straps. The end form was a metal plate punched so that the three stressing studs and the four longitudinal bars could be threaded through the plate and pulled up tight with a nut to prevent displacement during pouring. The base of the forms was carefully lined up and bolted to the floor.

Electing to use high-early-strength, air-entrained concrete, and to steam cure his beams, the contractor set up a schedule of casting two beams every Monday, Wednesday and Friday. This schedule was interrupted many times, but in the end proved entirely satisfactory. Early in the morning of the scheduled days a crew of about five men removed the side forms and jacked the beam about 3 in. sideways so that when stressed it would not bind in the forms. This operation took about a half hour. The stressing consumed another 30 minutes. With the help of an overhead crane it took about 10 min to remove and stack the beam. Resetting of the forms and installation of reinforcement and strands took about two hours, and the pouring and vibrating another hour and a half, so that after the work was well organized, and barring delays, the entire operation, which started at 7:30 in the morning, could be completed about noon. The crew then went back to its regular work of making pipe for the next day and a half.

The minimum interval between pouring and stressing was about three hours short of two days, and the cylinder tests indicated a strength well above the 3,300 psi required by the specifications. Seven-day cylinder tests on the beam concrete gave an average of 5,710 psi for 53 tests after normal curing, and an average of 6,330 psi for 31 tests after steam curing.

From a one-bag mixer set up beside the beam, the concrete was chuted into a buggy and then shoveled into the forms. There was only ¾ cu yd of concrete in a beam and the dimensions were so small that it literally had to be placed with a teaspoon. By using an internal vibrator and tapping the form, 50 well-formed beams without rat holes were obtained, with only the normal amount of air holes on top of the tee. The first beam was poured on Jan-



Prestressed beams were transported by truck to site, where they were unloaded by crane (below), and placed in bridge (lower right). Concrete beam was suspended from steel I-beam at third points by means of handling stirrups cast into concrete beam. Top view of deck, with reinforcement in place, is seen at right.

uary 30, and the last on April 11, 1952.

The stressing was a novel experience for the uninitiated. Two 30-ton brand new Simplex jacks were set in accordance with instructions. There were unexpected difficulties, however, when the two bottom strands were to be stressed together with a load of 13 tons each. One of the strands at the 13-ton load did not elongate the required 1 11/16 in. Jacking at both ends of the beam soon remedied that. The paper had been wrapped a little too tight on the strand and had caused bonding. It was tight only at one point because the sum of the measurements at both ends gave the required elongation. Next, when a strand was extended, the stud came out encased in concrete, and an hour was spent removing this so that the nut could be screwed up. The contractor's cardboard sleeve, covering the fitting, had moved during the vibrating and had become filled with concrete. After this experience, the sleeves were wired to the rods and the ends were calked with tissue paper.

The next difficulty was an excessive elongation—unexplained until someone noticed that the 7-wire strand was unwinding. This trouble was overcome by holding a homemade wrench on the end nut of the ram to prevent rotation.

Beams Restressed after Setting

Being inquisitive, we decided after we had stressed the beam that we would release the loads. To our amazement the nuts did not set by about one-eighth of an inch. The nuts were again set against the plates, the strands restressed to 13 tons, and the elongations were again about 1 11/16 in. From then on, each strand was stressed, released, and then restressed. Before each stressing the nut was set against the plate and the loss and extension measured.

The record kept on 150 strands indicates that some variation in individual strands is to be expected, but the averages show uniformity of wire and accuracy in the gage readings on the jacks. The average camber induced in the beams by the stressing was 1/4 in.

As we were not satisfied with stressing twice initially, jacks were attached to the strands of beams Nos. 5, 6, 10, 25, 27, and 31 and it was found that the nuts had to be pulled out from 1/8 to 1/4 in. from the bearing plates to put 13 tons into the strands. As a result, after all the beams had been stressed and had had time to set, each one was restressed in the shop to 13 tons. The measurements are tabulated in Table I. On the restressing, the gage was read when the nut left the plate, and when the load had reached 13 tons the distance of the nut from the plate was read.

The elongation measurements, judged to be more reliable, show that there was an average loss in stress of about 17 percent in these beams, whose ages varied from 24 days to a little over 3 months. The losses did not seem to increase over the three-month period, indicating that they took place soon after the beams were stressed. While the measured changes in the length of the beams were erratic, it would seem that the average shortening of the beams over this period was about 1/8 in. If this is so, there was a loss of 7.5 percent due to shortening of the beam and a loss of 9.5 percent due to creep in the strands or to other causes at present unknown to us. As a final seal of approval on the beams and to discourage further testing, the nuts were spot welded to the end plate and the beams accepted for installation.

On June 12, 1952, the beams were loaded on four trucks and delivered to the site, there to be unloaded by a crane directly from the trucks to the bridge seat.

The center beam was placed first. Because of minor irregularities, the subsequent beams laid up about 4 in. greater in width on each side of the center line than had been planned. The only problem in erection was in lining up the beams so that the transverse rods could be placed, but this was accomplished by threading one pilot rod through as the beams were set.

After the beams were placed, the remainder of the construction was routine. Side forms were erected for the roadway slab; the joints between the flanges of adjacent beams were plastered to insure tightness; and the cast-in-place concrete was poured in one day. The encasement of the sidewalk beams and the sidewalk slab were then poured; the deck slab was protected with 5-ply membrane waterproofing; 2 1/2 in. of bituminous concrete surfacing was applied; and the bridge was ready for travel.

Three Test Beams Broken

As a part of a broad research program with the Massachusetts Institute of Technology, three extra beams were cast for testing to destruction in M.I.T.'s laboratory. To simulate the actual field conditions for these tests, a cap of 3,750-psi concrete was poured around the stem of the beams, making in effect a rectangular composite beam 17 in. deep and 12 in. wide. Also, to simulate the design load, a single 6,800-lb load was applied by the testing machine to the third points of the beam, this

TABLE I. Cables Restressed to Restore 13-Ton Tension (Elongation in Inches)

Beam No.	Left Cable		Right Cable		Top Cable		Beam No.	Left Cable		Right Cable		Top Cable	
	Exist. Tension Tons	Elong. at 13 Tons	Exist. Tension Tons	Elong. at 13 Tons	Exist. Tension Tons	Elong. at 13 Tons		Exist. Tension Tons	Elong. at 13 Tons	Exist. Tension Tons	Elong. at 13 Tons	Exist. Tension Tons	Elong. at 13 Tons
1	13	0	12 1/2	0	11	2/16	27	10	3/16	10 1/2	4/16	11	4/16
3	10	6/16	9	4/16	9	4/16	28	10	6/16	10	5/16	10 1/2	4/16
4	10	5/16	10 1/2	4/16	11 1/2	3/16	29	10	4/16	10	5/16	11	4/16
5	10	4/16	10 1/2	4/16	11	2/16	30	10	5/16	10 1/2	5/16	10 1/2	3/16
6	11	3/16	10 1/2	6/16	11	3/16	31	10	3/16	10	6/16	10 1/2	5/16
7	12	1/16	10	3/16	10 1/2	5/16	32	10	5/16	10 1/2	3/16	10 1/2	4/16
9	10 1/2	6/16	10 1/2	5/16	10	5/16	33	10 1/2	3/16	10	4/16	11	3/16
10	10 1/2	5/16	10 1/2	5/16	10	4/16	34	10	5/16	10	4/16	10 1/2	5/16
11	9	7/16	10	5/16	9	6/16	35	10 1/2	5/16	10	5/16	11	4/16
12	10	7/16	10	6/16	10 1/2	5/16	36	10	6/16	10 1/2	5/16	10 1/2	5/16
13	10	5/16	10	5/16	11	4/16	37	9 1/2	5/16	9 1/2	6/16	11	4/16
14	10 1/2	4/16	11	5/16	10	5/16	39	10	6/16	10	6/16	10	5/16
15	9 1/2	6/16	11	4/16	10 1/2	5/16	40	10	5/16	10	5/16	10	2/16
16	10	4/16	10	5/16	11	3/16	41	9 1/2	3/16	10 1/2	4/16	10	6/16
17	10 1/2	5/16	11	5/16	10	5/16	42	10	5/16	10 1/2	4/16	11	3/16
18	9 1/2	6/16	10	6/16	11	4/16	43	10	5/16	10 1/2	3/16	11	2/16
19	9 1/2	6/16	11	4/16	10	4/16	44	10	6/16	10	4/16	10 1/2	4/16
20	10	5/16	10 1/2	5/16	11 1/2	1/16	45	9 1/2	5/16	10 1/2	5/16	11 1/2	3/16
21	10	6/16	10	5/16	11	2/16	46	10 1/2	4/16	10 1/2	4/16	11 1/2	1/16
22	10	5/16	10 1/2	5/16	10	5/16	47	10 1/2	5/16	10	5/16	10 1/2	4/16
23	10	7/16	10 1/2	5/16	10 1/2	4/16	48	9 1/2	5/16	10 1/2	5/16	10 1/2	4/16
24	10	5/16	10 1/2	4/16	11	4/16	49	9 1/2	6/16	9 1/2	6/16	10 1/2	4/16
25	11	4/16	10	5/16	10	5/16	51	9	5/16	10	4/16	10 1/2	4/16
26	11 1/2	4/16	10	6/16	11	3/16							

being the equivalent of the live load plus impact, plus the weight of the wearing surface. The concentrated load equivalent of the live load plus impact was 6,110 lb. The program called for the prestressing cables in two of the beams to be unbonded and for those in the third beam to be grouted.

The test beam No. 1 was the second beam cast, test beam No. 2 the thirty-eighth, and test beam No. 3 the fiftieth. The cables in the latter were grouted. Grouting was accomplished by encasing the strands in a thin-gage $1^1/_4$-in. octagonal tube before placing, and by filling the area between the strand and the casing with cement fondu after the stressing had been applied. The grout was inserted by gravity. A sample cut from the broken beam proves the success of that operation.

Beam No. 1 was tested on February 23, 1952, when the concrete in the beam was 15 days old and that in the cap 9 days old. The first crack in the prestressed beam appeared near the diaphragm under a load of 13,000 lb. From then on, well distributed vertical cracks appeared in the middle third of the beam. Failure occurred at 26,300 lb when the beam had deflected a total amount of about $7^1/_4$ in.

Failure was initiated by excessive tensile elongation. As the resulting large cracks progressed upward, the compression area was reduced until finally a compression failure occurred in the reduced compression zone. This took place at a point about 20 in. from midspan. The cracking load was equal to the dead load plus 2.15 times the sum of live load and impact, and the ultimate load was equal to dead load plus 4.3 times the sum of live load and impact.

Beam No. 2 was tested on April 5, 1952, when the concrete in the beam was 12 days old and that in the cap 9 days old. The first crack appeared 13 in. from the center diaphragm at 12,500 lb. The cracks from then on were vertical and distributed as in beam No. 1. Failure occurred at 26,000 lb, at which time the beam had deflected $6^3/_8$ in.

The failure was similar to that of beam No. 1 and took place at the same point. The cracking load was equal to the dead load plus 2.04 times the sum of live load and impact, and the ultimate load was equal to the dead load plus 4.25 times the sum of live load and impact.

Beam No. 3 was tested on May 3, 1952, when the concrete in the beam was 22 days old and that in the cap 19 days old. The cap concrete on the day of the test had a strength of 5,600 psi and the beam concrete a strength of 6,350 psi. The first crack appeared about 13 in. from the center of the beam under a load of 13,500 lb. Subsequent cracks were vertical but were distributed beyond the third points; they were more numerous, but did not open as wide as those in beams Nos. 1 and 2. Failure occurred at 30,800 lb, and at that time the beam had a deflection of 6.5 in.

The failure was similar to that in the other two beams, but took place under one of the applied loads. The cracking load was equal to the dead load plus 2.2 times the sum of live load and impact, and the ultimate load was equal to the dead load plus 5 times the sum of live load and impact.

In addition to the usual SR4 strain gages installed on the outside of the beams, an internal cylindrical compression gage was used. It consists of two $1^1/_2$-in.-dia end plates separated by a ring on which four SR4-type A7 strain gages are attached. A thin metal band encased the ring, sealing the interior of the gage. While these gages had been carefully developed and calibrated they had never been used before on a full-scale test. Four gages were installed in beam No. 3, as shown in Fig. 4.

If the gages are correct, there were compressive stresses in this beam in excess of those calculated. The calculated stress at gage No. 2 was 750 psi, the gage stress 900 psi. The calculated stress at gages Nos. 3 and 4 is 640 psi and the gage stress 1,150 psi. Shrinkage of the cap concrete may have induced the additional stress in the top gage.

Field tests were run on the completed structure, which had a final depth of 18 in., on August 5, 1952. A single 20-ton truck was used having a wheel base of 13 ft 10 in.; a spacing of 7 ft 6 in. center to center of back dual wheels, and 6 ft 3 in. center to center of front wheels. The total load on the rear axle was 22,225 lb and that on the front axle, 18,175 lb.

The structure was fully instrumented with both SR4 strain gages and internal compression gages, and for deflection a measuring device composed of a plunger deflecting a small calibrated aluminum beam was used. The magnitude of maximum tensile and compressive stress changes due to the application of the live load, as obtained from gages at the extreme top and bottom fibers of the bridge, was about 100 psi.

Prestressing Here to Stay

My humble conclusion on prestressing, based on our experience on the Danvers bridge and on five other bridges which we have now under

(*Continued on page 108*)

FIG. 4. Internal stress gages (SR4, type A7) in test beam No. 3, used for first time on full-scale test, showed higher-than-calculated compressive stresses.

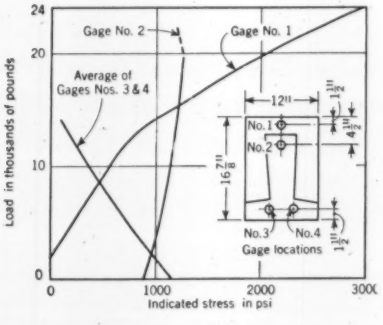
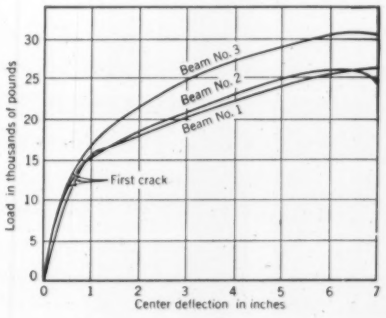
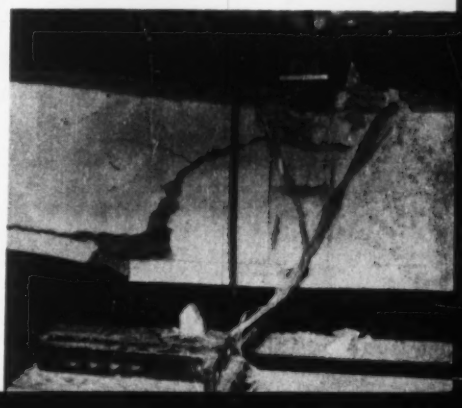
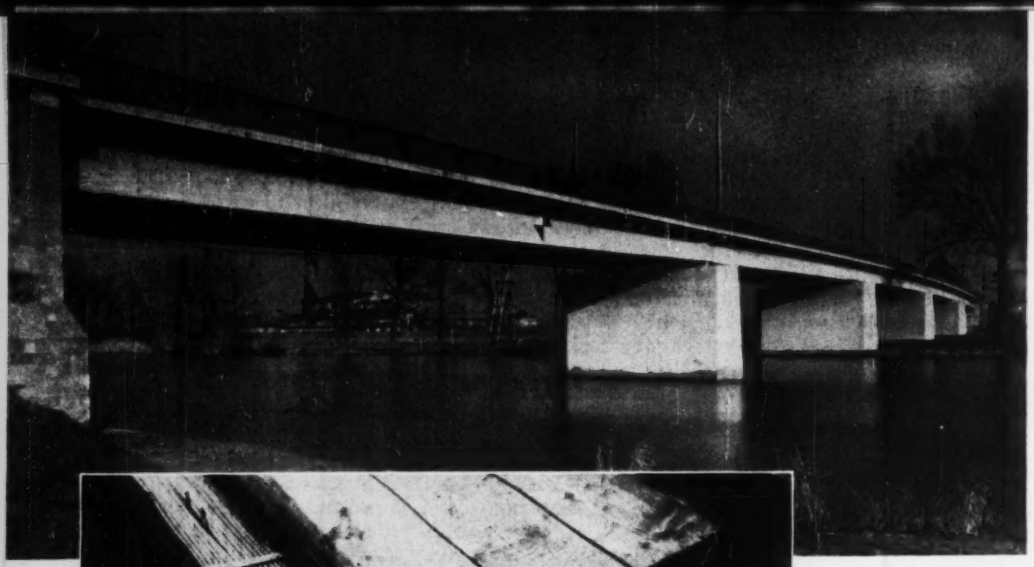


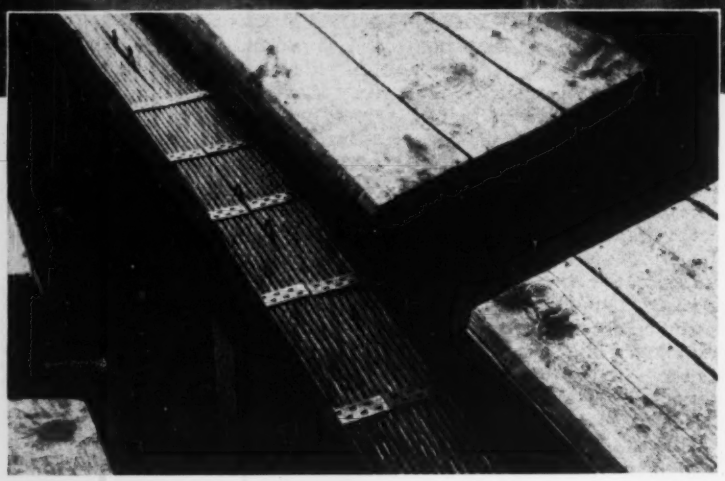
FIG. 5. Load-deflection curves for three test beams showed first cracks at approximately twice live load. Grouting in of prestressing cables in beam No. 3 increased its stiffness and ultimate strength.



Test beam No. 3 fails in Massachusetts Institute of Technology laboratory under ultimate load equal to dead load plus 5 times sum of live load and impact. In this beam prestressing rods were grouted in. For comparison between its behavior under test and that of two test beams with ungrouted rods, see Fig. 5.







Above
Completed bridge has simple, clean lines. Total length is 748 ft and maximum span is 141 ft.

Left:
Concentration of prestressing forces, accomplished by placing large number of cables in metal sheath, minimizes anchorage problems. Entire structure can be stressed in one operation.

Continuous bridge girder prestressed

Although in the United States there is a certain reluctance to make beams continuous over a number of piers, in Europe experience has shown that bridges, particularly those with shallow beams, are not much influenced by even extremely irregular settlement of the piers. It is true, however, that in the case of prestressed concrete, slightly more care is necessary than with steel beams, but the possibility of elastic and plastic deformation is rather great. In case very irregular settlement of the piers is expected, the diaphragms above the piers may be constructed strong enough to allow equalization by hydraulic jacks besides the bearings. This method is less expensive than the installation of joints, which require continual maintenance.

One of the most striking examples of the principle of continuity in a prestressed bridge is the Neckar Bridge at Neckargartach, which has five spans, consecutively of 137 ft, 141 ft, 141 ft, 141 ft, and 137 ft (Fig. 1). The overall length of the bridge, including abutments, is 748 ft. The supporting girder, 5 ft 9 in. in height, consists of two hollow boxes (Fig. 2), which are joined by two deep, one-piece, transverse stiffeners in each span, to give torsional stiffness to the whole cross section.

On one of the center piers the girder is fixed; on all the other piers it can move longitudinally through pendulum joints 4 ft $3^1/_4$ in. deep (Fig. 1). Provision for ample movement in the longitudinal direction is particularly important with prestressed concrete bridges because of temperature changes, elastic compression, creep, and shrinkage.

Prestressing Cables Placed

The prestressing cables run through the $13^1/_2$-in.-thick webs of the beams, and at the abutments are anchored in sling fashion around two movable stressing blocks. The entire prestressing force necessary has been established as 8,960 kips. Reduction of the prestressing force by shrinkage and by creep of concrete and steel has been computed to be 13 percent as a maximum. With this construction, the prestressing force specified is so ample that, even after losses by creep and shrinkage, there is a compressive stress of 140 psi in the concrete under full traffic load.

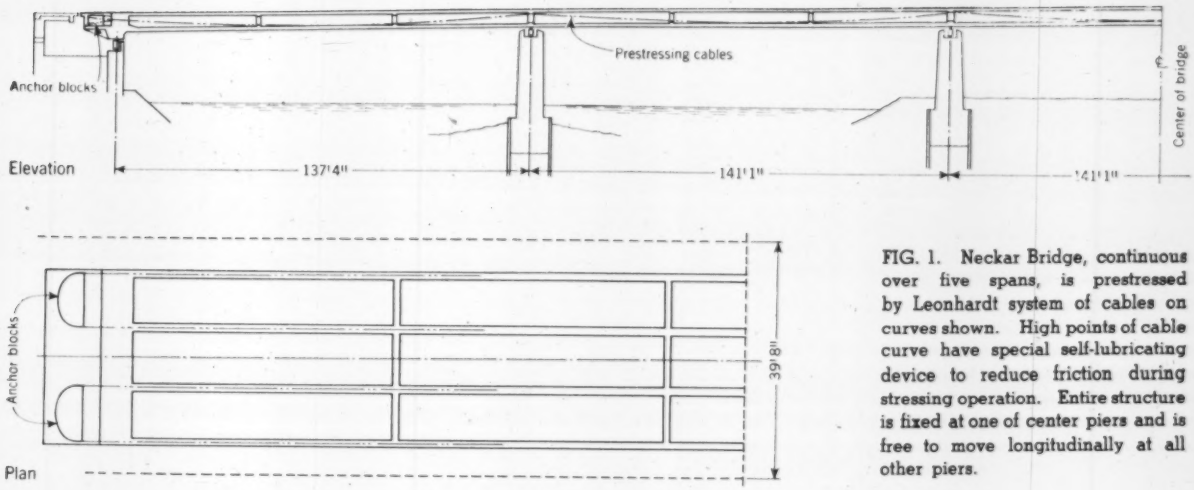


FIG. 1. Neckar Bridge, continuous over five spans, is prestressed by Leonhardt system of cables on curves shown. High points of cable curve have special self-lubricating device to reduce friction during stressing operation. Entire structure is fixed at one of center piers and is free to move longitudinally at all other piers.

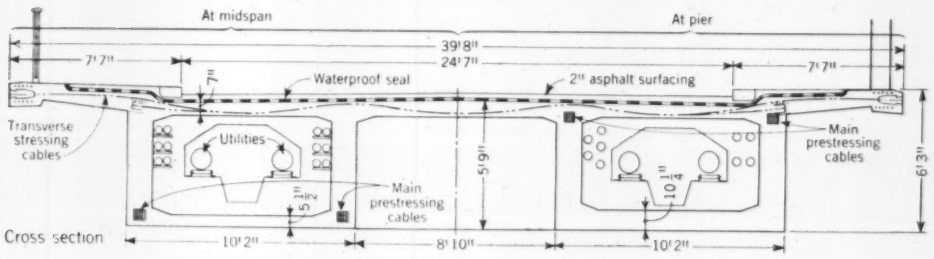


FIG. 2. Concrete box girders supporting bridge have prestressing cables concentrated in metal sheaths running through walls, as shown in photo on facing page. Interior of boxes provides space for carrying utilities.

in a single operation

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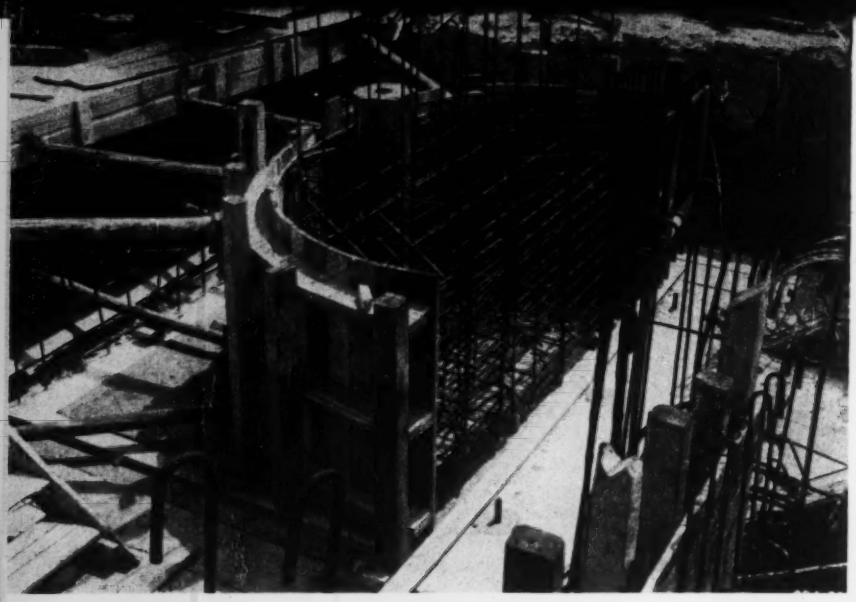
In a vertical direction the position of the prestressing cables is determined by the moments. In the center of the span the cables are at the bottom of the girder, and at the piers they are at the top. At the piers, a circular curvature on as short a radius as possible is used, so that the normal forces which act there due to the change of direction in a downward direction can flow directly to the bearings on the pier. In the spans, usually a parabolic but pologonal curvature is used so that there will be only a few locations where the normal forces of the cables will act in an upward direction. With the normal forces the prestressing produces moments which counteract the dead weight of the bridge. The moments can be influenced to a great extent by the correct choice of curvature for the stressing members. Also, the total moments are influenced by the vertical position of the stressing force at the abutments.

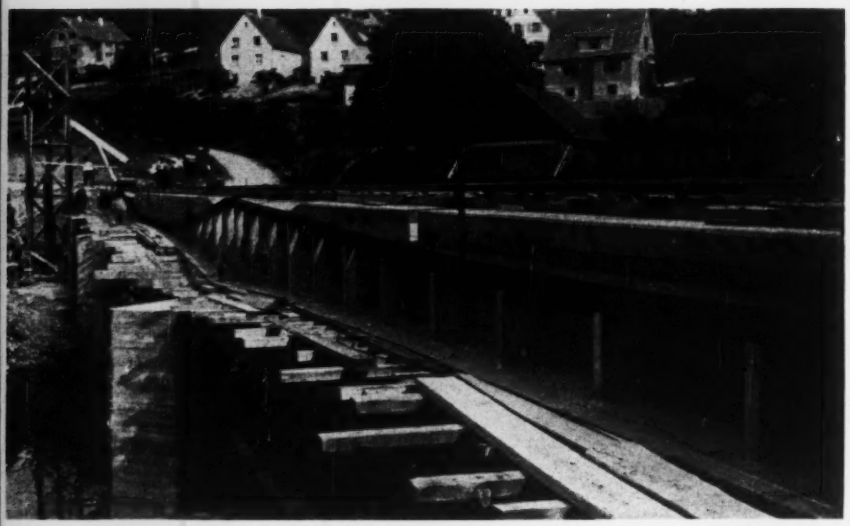
To reduce friction in the cables at the curved points, cold-rolled hard sheet-metal strips with an intermediate layer of paraffin are inserted between the cable and the cable casing. In addition the sheet-metal casing is reinforced with a bent steel sheet, about $^3/_{16}$ in. thick, in order to transfer the normal pressure. By careful execution, friction coefficients can be held to 0.07 at best and 0.25 at worst. The actual friction had been found by measuring the longitudinal displacement of the stressing cables at various intermediate points through small openings in the concrete. The compression in the concrete was also measured.

Stressing Force Checked

In order to provide the computed stressing force in the middle span without losses due to friction, the jacking force at the ends of the cable will temporarily exceed the specified force—the amount depending on the observed friction—until the desired stress is reached at the center. Then the stressing force at the ends is slacked off and the procedure repeated for the next span. The stressing force in the jacks has to be checked continuously by manometers in order to avoid an excessive stress in the ends of the cables. With this procedure it can be safely assumed



Heavily reinforced anchor block is poured on abutment at each end of bridge. Prestressing cables passing around block are all stressed at once by hydraulic jacks between bridge slab and block.



Metal sheath which holds prestressing cables is supported in final position on blocks. Pipe in foreground is for grout intrusion.

Curvature of metal sheath over piers is as short and circular as possible. Downward thrust of cables is taken by piers.

that the stressing force considered in the calculations will at least be obtained on the whole length.

The blocks against which the jacks react are cast in simple wooden forms, with a comparatively small amount of reinforcing, on smooth-surfaced cantilevered concrete slabs at the abutments. The free ends of the stressing strands run through small metal tubes into a vertical opening in the anchor block, where they are embedded in concrete to hold them perfectly. When the cables are stressed, the entire anchor block is compressed heavily in two directions. However, the block is shaped in plan in such a way that the resultant force of the strands and jacks remains somewhere in the kern of the stressing block.

Diesel Engine Lays Strands

After the anchor blocks are cast, the cable casings are placed and supported by concrete blocks or welded steel bars in the correct vertical position. Then the strands, with their spacers, can be laid. In this particular case, the laying operation was performed by a diesel engine on raised tracks pushing a small carriage with two small horizontal reels over which the strand-slings run. In this way the strands were automatically placed in the sheet-metal boxes and had only to be adjusted manually.

Using this method, the cables for the whole bridge were placed in four days. Considering how much labor is necessary to lay the heavy bars of an ordinary five-span concrete beam bridge of the same spans, the economy realized by this prestressing method is readily appreciated. After

Stressing cables are placed in sheath mechanically by diesel powered carriage. Final position of cables in sheath is adjusted manually. By this method, cables for whole bridge were placed in four days.

the strands are laid, the box is closed, the reinforcing placed, and the rest of the form work erected.

The bridge was also prestressed transversely. The road slabs had been computed with fixed ends. The degree of fixity is high because of the torsional rigidity of the hollow boxes. A calculation of the box as a closed frame proves also that this assumption is correct.

Transverse Prestressing

Stressing members of 52-kip stressing force, placed 3 ft apart, are sufficient for the transverse prestressing when placed continuously across the three parts. They have also a slight parabolic sag in each span of the slab. So-called "Leoba" stress members were used. In these members 12 wires of 0.208-in. diameter, in two layers of 6 each, run through a flat metal tube. This arrangement of the wires results in a much lower friction than the usual circular arrangement. In addition to this, the Leoba stress members are anchored at the ends in such a way that after stressing and injection with mortar, no steel parts extend through the surface of the concrete. Transverse prestressing was done before longitudinal prestressing.

An aluminum sealing membrane and an asphalt surfacing 2 in. thick were laid on the prestressed roadway slab.

Numerous cables and pipelines to supply adjacent parts of the town can be laid through the hollow boxes without detracting from the appearance of the bridge.

Generally, the prestressing of such a bridge takes place in three stages. It is started three or four days after the gaps above the piers are concreted as the last sections, but only very lightly, in order to avoid cracking due to losses in the heat of hydration or early shrinkage in the very extended concrete structure. After three weeks, in the second stage, the steel is stressed sufficiently to make the bridge carry its own weight. Thereupon the scaffolding is lowered to make the dead weight show its full effect, and then full stressing is effected.

During stressing all hydraulic jacks at both ends of the bridge are connected together so that the prestressing force will be exerted evenly at both ends by one high-pressure pump.

As compared to prestressing with a number of small stressing elements, this mechanical stressing procedure saves much labor. Furthermore, it is advantageous to have the prestressing force distributed symmetrically and evenly across the entire width of the structure.

The jacks had to move each stressing block an average distance of 25 in. in the longitudinal direction, which represents the elongation of the cables over the entire length of the bridge.

After each stressing stage, precast concrete blocks were inserted in the spaces between stressing blocks and bridge ends, with mortar joints to secure an equal pressure on the whole block area.

Final Concreting and Grouting

After prestressing was completed, the stressing blocks and jack openings were covered and filled with concrete. This concrete is strongly reinforced in both directions to make it practically crack proof, as it receives no prestressing.

In grouting the sheet-metal cable cases, injection tubes are provided at the low points and air escape tubes are set at the highest points above the piers. During filling, the concrete mortar rises from the lowest point, displacing the air.

The concrete mortar has a low water content in order to insure a good consistency and a strong bond between the prestressing cable and the surrounding concrete. This bond is absolutely necessary to realize the specified factor of safety. For this purpose, the 7-wire strand gives a much better bond than smooth wires. In addition, the sheet-metal case may be roughened to provide a grip there also. It has been shown on beams 66 ft long that the bond of the strand cables is so good that the required factor of safety is exceeded.

Material required for this structure amounted to 1.64 cu ft of concrete per sq ft, 5.53 lb of stressing cable and 8.4 lb of standard steel per sq ft.

Today in Germany such prestressed concrete bridges are from 15 percent to 25 percent cheaper than steel bridges of the same depth and length, without considering maintenance. More than 40 continuous girder bridges have been built by this method to date.

The bridge here described was built in 1951 by two companies—Ludwig Bauer of Stuttgart, and Heinrich Butzer of Dortmund—under the supervision of the engineers Dr. Daser and W. Stoehr of the City of Heilbronn. Prof. Dr. Deininger of the engineering faculty of the University of Stuttgart and the author acted as consultants.

Prestressing warrants study to simplify construction methods

MAXWELL M. UPSON, M. ASCE

Chairman of the Board, Raymond Concrete Pile Co., New York, N.Y.

To the fraternity of engineers who have long dealt in concrete, prestressing is the answer to a fervent and oft-repeated prayer. The dream of a homogeneous concrete with real tensile strength has come true. From the beginning, all of us who have built with reinforced concrete have been conscious of its serious drawbacks. Steel and concrete differ so markedly in physical characteristics that it has heretofore been difficult to work them together as a homogeneous whole.

It is well known that in long members the shrinkage in the concrete during setting may place the steel under a compression amounting to well over 12,000 psi. Since this compression in the steel must be removed before its normal tension function can become effective, it is natural that the concrete, with its low elasticity, will not only lose its tensile strength but also develop cracks. If later overloading occurs, these cracks naturally become serious and result in steel oxidation and concrete disruption.

Prestressed concrete with its qualities of flexibility and freedom from deteriorating cracks is the answer to some of these drawbacks. As a structural material it is excellent, but is it cheaper? Is it possible to develop construction methods to make it economically competitive? I think so.

Shortly after the writer became interested in concrete, over 50 years ago, the National Association of Cement Users was formed. Its first convention was held in January 1905, in Indianapolis, Ind., with John P. Given of Circleville, Ohio, as chairman. This organization later became the American Concrete Institute. It was stimulated and guided by many able and distinguished promotors of the industry, including Richard L. Humphrey, M. ASCE; Dr. William K. Hatt, M. ASCE; Henry C. Turner, M. ASCE; William P. Anderson; Dr. S. C. Hollister, M. ASCE; Harvey Whipple; and many others. I also am proud to have had a part in this effort. Mr. Humphrey gave a large part of his time to the organization during its first ten years.

Thus the inherent weakness of reinforced concrete previously referred to has been the subject of thought and experiment ever since 1905. However, it was not until a trip to Europe in 1937, that my attention was called to Mr. Freyssinet's use of the elastic characteristics of wire to take care of the shrinkage and plastic flow of concrete. This, I felt, was a real solution to the problem.

On this and subsequent visits to Europe my studies of the detailed development of prestressing disclosed that although the product was right, its production in the United States would be limited until economic methods of operation could be devised. Fussy and detailed operations possible with labor at 50 cents an hour become completely uneconomical at American rates four to six times higher. Utilizing European methods, our labor costs would much more than absorb the saving in steel and concrete. Even when a new product is markedly better, experience has shown that it is difficult to introduce it if its cost is materially higher.

Observation leads me to believe that as yet most of us have little knowledge of the true merits of this new method of combining steel and concrete. Much research and many tests must be carried out before engineers will be completely convinced of the advantages that accrue in the prestressing of concrete. And, of course, such information must be available before designs will be modified to realize these advantages.

The truth of this statement is evidenced by the fact that some European designers allow for a tensile stress in the concrete varying from 400 to more than 1,200 psi. It is easy to recognize the weight reduction and economy of materials possible from a wide adoption of such allowances.

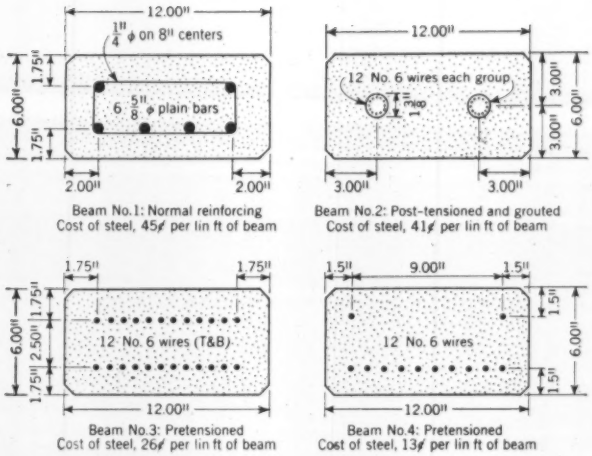
With a view to setting forth the progress that has been made in these efforts during the past fifteen years, by the organization with which I am associated, I give you the results of some of our experiments.

To illustrate the savings in both quantity and cost of materials that accrue from the use of prestressing, four types of sheetpiling were designed with different reinforcing and different types of prestressing.

Twelve beams, three each of the four types shown in Fig. 1, were subjected to bending tests in the engineering department of Tulane University. These were tested as simple beams over a free span of 12 ft with the loads applied at the third points. Strain-gage readings were made at various places, and the strain and deflection in each beam were measured. The relationship between the load and deflection for each of the four types is shown in Fig. 2. The curves represent the average of the three specimens of each type.

Based on present prices, the cost of the steel for the standard reinforced beam is approximately 45 cents per lin ft. In Beam No. 2, where the center reinforcing uses cables that are more expensive than plain wire, although much less costly to install, the cost is reduced to 41 cents per lin ft of beam. The 24 pretensioned wires in Beam No. 3 bring the cost down to 26 cents per lin ft of beam. In beam No. 4, the distribution of the wires on the basis of the stress to which the pile is to be subjected, brings the cost down to 13 cents per lin ft of beam.

All the above is set forth for two

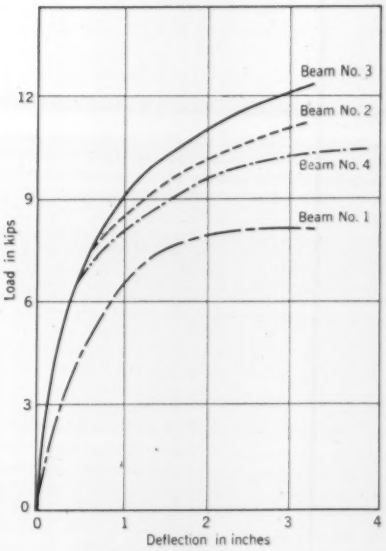

12.00"
1" / 4 ϕ on 8" centers
1.75"
6.00"
6 5/8 ϕ plain bars
2.00"
Beam No.1: Normal reinforcing
Cost of steel, 45¢ per lin ft of beam
12 No. 6 wires each group
3.00"
Beam No.2: Post-tensioned and grouted
Cost of steel, 41¢ per lin ft of beam
1.75"
2.50"
12 No. 6 wires (T&B)
Beam No.3: Pretensioned
Cost of steel, 26¢ per lin ft of beam
1.5"
9.00"
12 No. 6 wires
Beam No.4: Pretensioned
Cost of steel, 13¢ per lin ft of beam

Above:

FIG. 1. Concrete beams, reinforced as shown, were tested to determine strength-cost relations. Cost does not include placing of steel. It is to be noted that most expensive beams from standpoint of steel cost had lowest ultimate strength and showed greatest deflections (see Fig. 2, right). All wires of prestressed beams were M.B. oil-tempered, with a yield point of 170,000 psi, and an ultimate strength of 210,000 to 220,000 psi. Initial prestress was 150,000 psi. The $^5/_8$-in. reinforcing bars were standard.

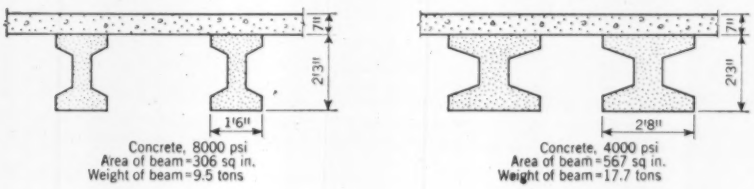
Right:

FIG. 2. Strength of four test beams is compared.



Left:

FIG. 3. Advantages of using high-strength concrete in prestressed beams for 60-ft span supporting ordinary concrete slab, under H20-S16-44 highway loading, are shown in two diagrams. Prestressed beams at far left are designed for 8,000-psi concrete and weigh 9.5 tons each. Beams at left, designed for similar conditions but using 4,000-psi concrete, weigh 17.7 tons. This difference in weight could have a significant effect on handling costs.



purposes: (1) to show that the location of the reinforcing in prestressing is an important factor, and (2) to indicate the marked savings in steel costs which are within reach if economical methods of placing are developed.

The deflection curves indicate that beams Nos. 2 and 3, in which the prestressing is uniform throughout the member, show slightly greater strength than beam No. 4; but it is evident that No. 4, with half the amount of reinforcement of Nos. 2 and 3, is still very much stronger than the beam with standard reinforcing. It will be observed that no hooping was used in the prestressed members and that steel costs do not include the labor of handling and placing. With a proper set up of facilities, the labor cost of placing the wire per foot of sheetpiling will be less than that for making and placing the cages of standard reinforcing steel.

Already the importance of continuous adhesion of the tension reinforcement throughout the length of the beam, in order to attain maximum carrying capacity, is well recognized. Early European tests demonstrating this important truth were made by P. W. Abeles, M. ASCE, in his article, "Ultimate Resistance of Prestressed Concrete Beams," published in *Concrete Construction Engineering*, October 1951.

Here the grouted wires give more than a 75 percent increase in ultimate beam moment as against the ungrouted wires. Or, stating it another way, if the bonding is not adequate and continuous, and end anchorage alone has to be depended upon, the strength of the beam is reduced over 40 percent.

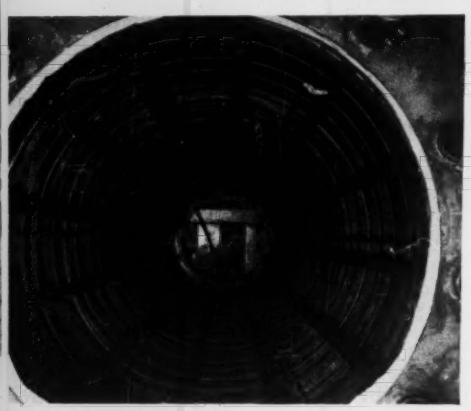
All the above facts point to the importance of having adequate bonding between steel and concrete. To attain this without peradventure, several important factors must have careful attention:

1. The relationship of the surface area of the steel to the cross-sectional area.
2. The method by which the bonding can be conveyed to the mass of the concrete.
3. Preservation of the relationship of the wires in position to one another so that they will be straight and all surfaces will be accessible to the concrete or grout.
4. A system of attaining this end which also permits the bending or curving of the prestressing elements.

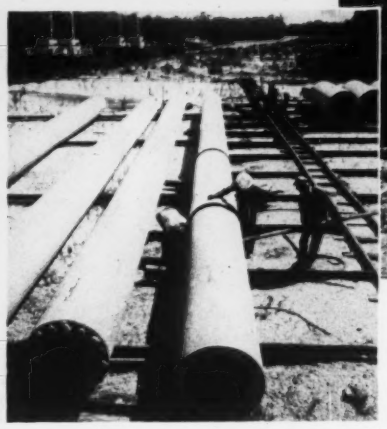
Another factor that is important in this new concrete technology is the advantage that accrues from high quality, high-compression concrete. Perhaps never before has concrete quality been so important. In European practice it is common to predicate a design on 6,000-psi concrete, and under special conditions in which weight is a factor, this strength may be increased up to 8,000 psi, or even 10,000 psi. Of course careful selection of materials, special screening and proportioning, together with controls, are essential to obtain any such strength. The advantages to be realized by using high-strength concrete in prestressed beams for a 60-ft span supporting an ordinary concrete slab for H-20 highway loading are clearly shown in Fig. 3.

A prestressed-concrete hollow pile provides a unit with a large bearing area and great surface friction area—all within practical handling and driving weights—and one which at the

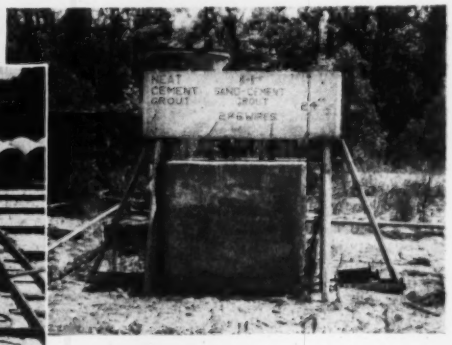
Right: End template is designed to provide smooth end surface. Projections around holes permit extra thickness of grout around wires at joints between pile sections.



Above: Prestressed piles are spun in forms such as those shown. Longitudinal mandrels are rubber-covered steel tubes which will be pulled out shortly after concrete has set so as not to develop too high a bond.



Below: To test bond of two No. 6 wires, load of about 8,600 lb was hung from wires bonded over length of 24 in. Stress in each wire is about 150,000 psi. Block has been suspended thus for almost two years. So far, no movement of the steel is evident.



Left: Full length of pile is secured by joining 16-ft hollow sections. Entire length is then fitted with stressing wires, compressed, grouted and driven. After grouting, pile can be cut off at any length.

same time has a section modulus that permits long columns. As it is never possible to predetermine pile lengths, a unit must be so built that it can be cut off at any length. This of course calls for a sure way of holding prestressing without end anchorages.

To attain that end, long and careful studies were necessary to ascertain the relationship of the cross-sectional area to the surface area of the reinforcing wire, and to make sure that the wires are kept approximately parallel and free from contact so that their surfaces are completely accessible to the grout. It is of course essential that the interior surface of the hole containing the wires be such as to properly grip the grout. Perhaps most important of all, it is essential to provide a grout that is not only workable but also of a character that will expand rather than contract when it sets.

Our tests running over a period of years have shown that these problems have been solved beyond question.

The practical use of these prestressed piles or caissons is demonstrated by the construction of an oil treating and control station in the Gulf of Mexico standing on piles 36 in. in diameter and 96 ft long.

These units are made of a number of centrifugally cast sections 16 ft long, held together by post-tensioned prestressing. By special methods the ends are spun perfectly, as if they had been turned by a lathe. The end surfaces are coated with a material that can be brushed on and which quickly develops a compressive strength greater than that of the concrete. This forms a joint which is watertight, simulating the ground joint flanges used in high-pressure steel-pipe construction. The end of the spinning structure which forms the end of the concrete section has a projection around each hole location which forms a recess in the concrete section. This provides extra grout around the wires at every joint, giving added assurance against possible infiltration of water or air.

In the handling and driving of thousands of lineal feet of these units, using hammers up to 10,000 lb, no evidence has been disclosed that there was any slippage of the prestressed wires. The prestressing in these units is attained by the use of a 12-strand No. 6 high-tension straight wire cable. The number of cables in each unit depends on the service to which it is to be subjected. It is evident from an examination of the tops of the piles that prestressing provides a toughness or resistance to abuse that has not been heretofore encountered with concrete.

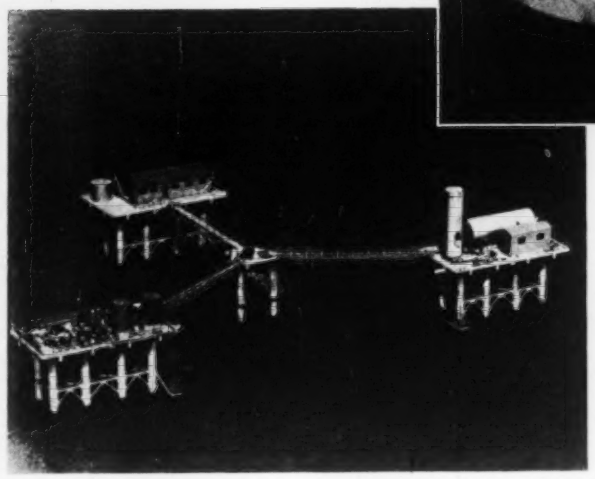
A series of careful tests have proved that, with proper grouting, 12-strand No. 6 straight wire cable will develop its breaking strength with an embedment length of from 24 to 36 in. The 36-in. length is required when the grout is in the early stages of setting, developing only about 2,000 psi. Less than 24 in. is required when the grout attains its full compression strength of from 5,000 to 7,000 psi.

As a continuous test of the adhesion of concrete to No. 6 wire, two wires were embedded in a block of concrete weighing about 8,600 lb. The 8,600-lb load gives a stress of about 150,000 psi in each wire. This block has been hanging from these two wires for almost two years. Thus far no movement of the steel is evident in the concrete, either in the block or in the support.

Elimination of the internal stresses that have heretofore been inherent in reinforcing concrete, combined with the tensile strength that prestressing develops, makes it possible to treat this material in much the same way that steel has been used for many years. The substitution of such members in structures exposed to weather and sea water has great advantages, not only in permanency and cost of maintenance, but sometimes also in original cost.

Offshore oil drilling islands are interesting examples of the application of prestressed units to solve some practical problems. In such construction prestressed piles varying from 36 to 54 in. in diameter have been used to form an important part of the structure, and prestressing is

Right: End of driven pile shows how little damage is done to prestressed section during driving. Pile pictured has received several thousand blows with 10,000-lb hammer.



Left: Prestressed piles support offshore oil-drilling rig. Piles for such structures have been designed for 25-ft depth of water with 35-ft waves, and 100-ft depth of water with 25-ft waves.

also used in the beams, floor construction and cross-bracing in order to save installation cost in addition to providing low maintenance. One such structure was designed for a 25-ft depth of water with a 35-ft wave, another for a 100-ft depth of water with about a 25-ft wave.

To test the resistance of prestressed concrete to the deteriorating actions of sea water, particularly in the areas between high and low tide, sections of prestressed piles were placed in the New York Harbor area in the year 1939. Not only were the piles exposed to high and low water in a vertical position, but short sections were also laid on the beach so that the ends, with the stressing steel exposed, were subjected to the same deteriorating action. The effect of alternate freezing and thawing was also involved. Naturally the wires which extended beyond the surface of the concrete rusted off, but to our surprise this rust stopped within $^1/_8$ to $^1/_4$ in. of the surface, and the steel embedded in the concrete beyond this distance was bright and clean, indicating that it had been completely protected.

These experiments, together with other observations, convince the writer that the old specification of requiring a cover of $1^1/_2$ to 2 in. of concrete over all steel in sea-water construction may be very much reduced if prestressing is used. The 13-year test shows that it may be safe to reduce this cover to as little as $^1/_2$ in. These characteristics open up a wide field for the use of prestressed concrete in structures such as tanks, boats, and scows.

To engineers who are familiar with anchorage devices for holding steel bars and wires under stress, it is unnecessary to explain the details of this phase of prestressing. All the devices in use seem to have merit and perhaps are a wise precaution under certain conditions, but the designer must keep in mind that the strength of the beam must be increased almost 75 percent to allow for the lesser moments that are inherent in the end-anchorage system. As soon as the continuous adhesion of the concrete to the steel ceases, the unit becomes an end-anchorage beam with less than two-thirds of its normal carrying capacity. With these facts before us, it is evident that in the interest of economy and saving in weight it is essential that the industry recognize the necessity of complete and adequate grouting of all the post-stressed steel elements.

Where the engineers we serve demand anchorages, we recommend a 2-in.-thick square plate drilled with a tapered hole to receive a tapered plug which locks the wires. This is economical and sure.

In the hollow piles or caissons previously referred to, the $1^3/_8$-in.-dia holes for the prestressing wires are formed by the use of rubber-covered steel tubes. These are withdrawn a few hours after the spinning of the sections. This produces a surface which gives excellent adhesion between the grout and the body of the concrete.

It is obvious that in long members the placing and removing of a core of this kind are expensive and in many cases impractical, particularly when curvature of the prestressed unit is desirable.

To meet this need our company has developed and patented a method for making a cable unit which not only produces the completely steel encased grouping of wires which are held in place by thimbles, but also provides flexibility and proper shouldering on both the inside and outside of the tubing so that longitudinal motion is impossible. This cable holds the wires in an even and uniform position so that all surfaces are exposed to the grout, and it also leads to great economy in handling and placing. The 12-wire cable can be put in position in the cage as easily as a 1-in. ordinary reinforcing steel bar and has approximately $2^1/_2$ times its reinforcing strength. In my opinion, the cost of placing steel in work of this kind, although coming to more per ton, will be considerably less per foot of unit.

Experience thus far indicates that we have in prestressed concrete a more durable and a more useful masonry material than has heretofore been available to the building industry. In consequence, it is incumbent on us as constructors of the future to devise means and methods to have its merits recognized.

As this discussion has indicated, our efforts thus far have been directed toward the finding of methods and procedures that are better and safer and will lead to cost reductions. The wider avenues of use to which this new child of the industry can be applied must be the subject of further consideration.

As has been stated, the use of reinforcement in concrete in a manner that will permit it to take high tension loads without cracking, and to avoid the normal cracks that develop through temperature changes, must be simplified to meet our labor rates if it is to attain wide acceptance in this country. It is my belief that Yankee ingenuity will solve these problems promptly and satisfactorily.

(This article is based on the paper presented by Mr. Upson before a Construction Division session presided over by A. H. Ayers, chairman of the Division's Executive Committee, at the Centennial of Engineering Convention.)

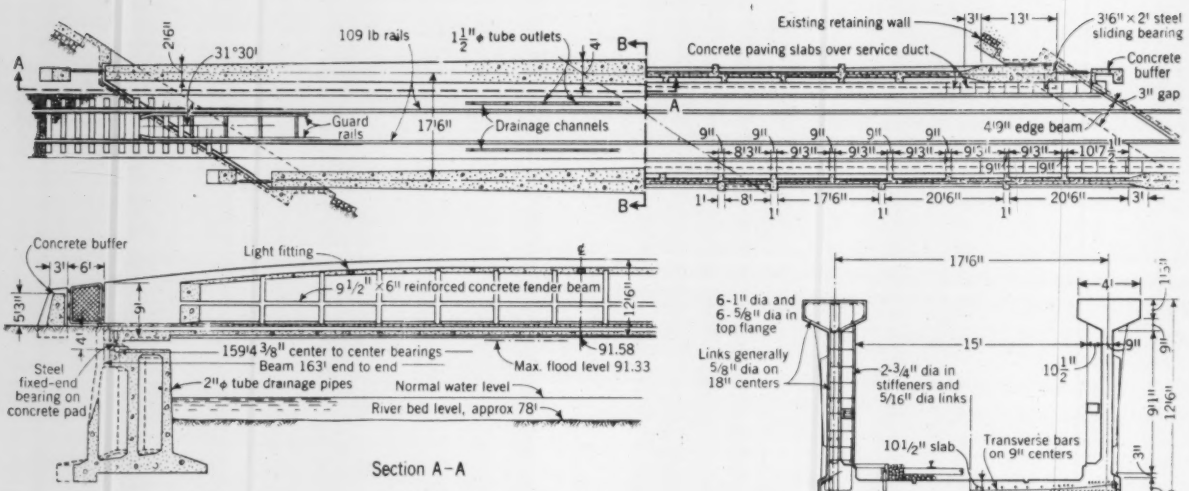
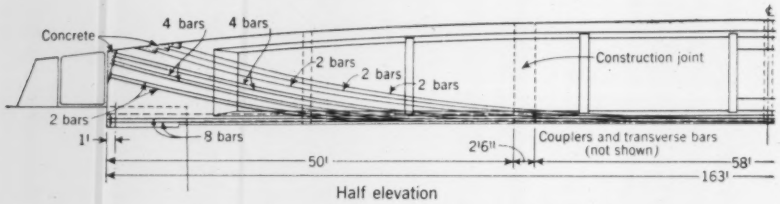

Existing retaining wall
Concrete paving slabs over service duct
3'6" × 2' steel sliding bearing
Concrete buffer
3" gap
109 lb rails
1½" ϕ tube outlets
31°30'
Guard rails
17'6"
Drainage channels
4'9" edge beam
Concrete buffer
Light fitting
9½" × 6" reinforced concrete fender beam
159'4 3/8" center to center bearings
Beam 163' end to end
91.58
Max. flood level 91.33
Steel fixed-end bearing on concrete pad
2" ϕ tube drainage pipes
Normal water level
River bed level, approx 78'
Section A–A
6-1" dia and 6-5/8" dia in top flange
Links generally 5/8" dia on 18" centers
2-3/4" dia in stiffeners and 5/16" dia links
101/2" slab
Transverse bars on 9" centers
Showing mild steel reinforcement, same for both girders
Showing 1 1/8" dia Macalloy bars, same for both girders
Section B–B

FIG. 1. Bridge of 160-ft span over River Don, England, is said to be longest prestressed railway bridge so far built. Its cast-in-place through girders are formed integral with floor slab. Prestress was applied by post-tensioning through fully bonded $1^1/_8$-in.-dia, high-strength Macalloy bars. Cross section B-B at center line shows, on left side, location of mild steel reinforcement in each girder, and on right side, of high-tensile-strength prestressing bars in each girder.



Left:
FIG. 2. Longitudinal cross section through bridge indicates how various prestressing bars are curved to follow parabolic profile.

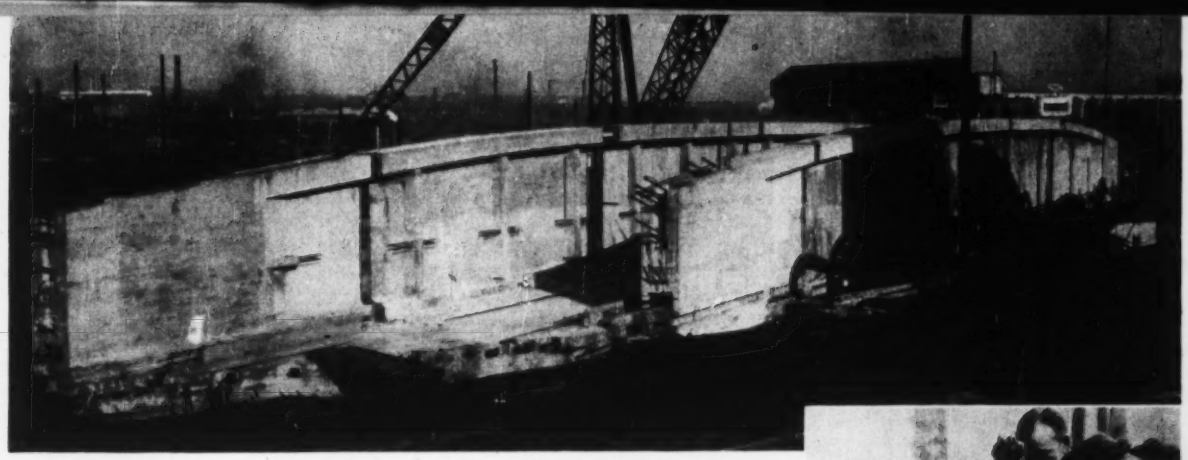
Ends of prestressing bars, some with bearing plates in place, protrude from concrete. Before prestressing, gaps in floor and girders will be concreted. Sway bracing at midspan has not yet been constructed. It was one of the last things to be done.

Prestressed

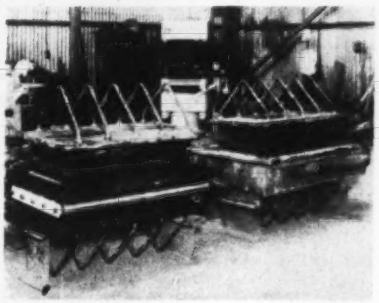
DONOVAN H. LEE, M. ASCE

Consulting Engineer, London, England

While the proportion of new highway bridges that have been built in England of prestressed concrete is high as compared to those of other materials, the use of prestressed concrete for railway bridges has only recently ceased to be exceptional. As there has been almost no new railroad construction in Britain in recent years, practically all rail bridge work has been for replacement of existing structures, mostly under conditions permitting very limited interruption to traffic. This limitation has generally not favored the use of prestressed concrete. Actually, however, prestressed concrete has



Above:
Girders, 160 ft long and 12 ft 6 in. deep at center, were cast in place integrally with floor slab, but in sections with gaps between. Prestressing bars were inserted in ducts, coupled together, and wrapped to prevent bond, before gaps were concreted.



Left:
Girders at their free ends are supported on combined rocker and roller bearing plate like assembly shown, with casing removed, at left. At their fixed ends they rest only on rockers.

Right:
Prestressing equipment includes jack on rubber tires in place in foreground. Note bearing plate and special threads and nuts on ends of high-tensile-strength bars. Average stressing time is 7 min per bar for deck but more for bars ending at tops of girders.

concrete railway bridge of record span completed at Rotherham, England

properties that make it particularly suitable for railway use. "Crackless concrete" combined with the low porosity of high-grade concrete make for durability and low maintenance. Also, with post-tensioned, fully bonded construction, the small variation in stresses in the prestressing steel between dead load and full load makes for a high resistance to impact and fatigue. This characteristic of prestressed concrete appears to be appreciated by British railway engineers, but an appreciation of the advantages of post-tensioning over pre-tensioning and of fully bonded over non-bonded prestressing steel may take longer to develop.

The new railway bridge at Rotherham (Figs. 1 and 2) is believed to exceed in span any railroad bridge so far built in prestressed concrete. It crosses the River Don at a considerable skew, necessitating a span of 160 ft.

When completed in January 1953, this bridge will carry a new industrial track connecting the British Railways (Eastern Region) with the plant of Steel, Peech & Tozer, one of the largest steel works in England. Because other alterations and extensions of the steel works were under way, the bridge was designed and built without particular urgency and without the necessity of maintaining rail traffic during construction.

A prestressed concrete design was chosen for minimum cost; it was estimated at 20 percent less than structural steel. The structure is designed for main-line loading, which provides for a 20 unit live load according to British practice, that is, 20 long-ton axles on 5-ft centers, plus impact. For this low-speed track, the impact factor was taken at 17.4 percent.

As the River Don in flood has been known to overflow its banks, and as

the bridge connects with the steel works tracks on both ends, the approach tracks had to be raised to keep the under side of the bridge above flood level. Even so, there is only a small clearance between the flood level of the river and the under side of the bridge. For this reason and because the bridge is located at a sharp bend in the river, it was believed that the use of a central pier would cause trees, branches and other debris to collect and obstruct the flow in time of flood. Further, the saving effected by using two short girders on each side of the bridge instead of one long one would not have been much more than enough to cover the additional cost of the central pier and the extra widening required to provide an equivalent clear waterway.

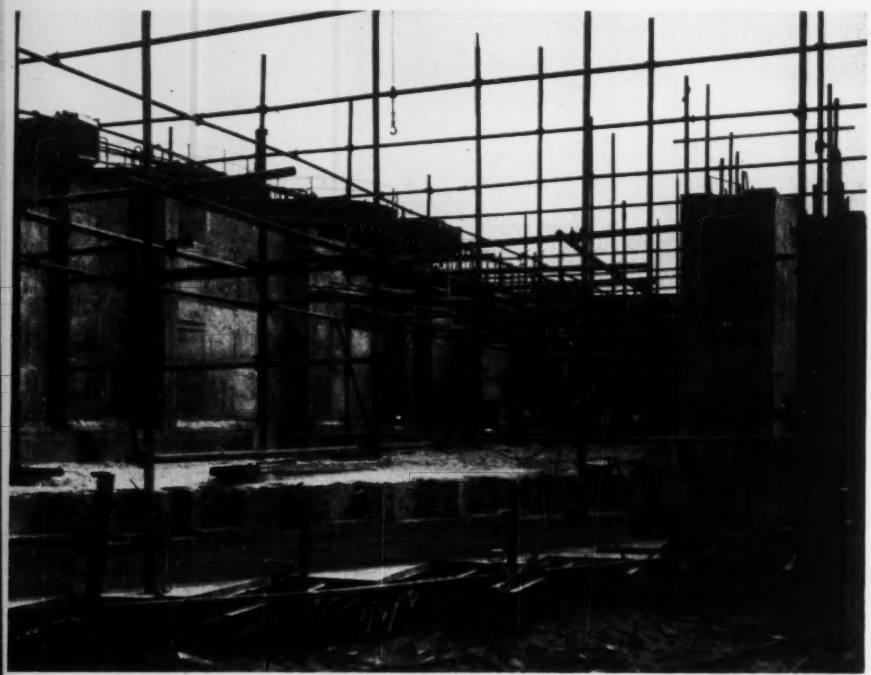
Accordingly two through girders were used, with a prestressed deck slab between them, the compression flanges of the girders being stiffened by concrete ribs placed at intervals on both sides of the webs.

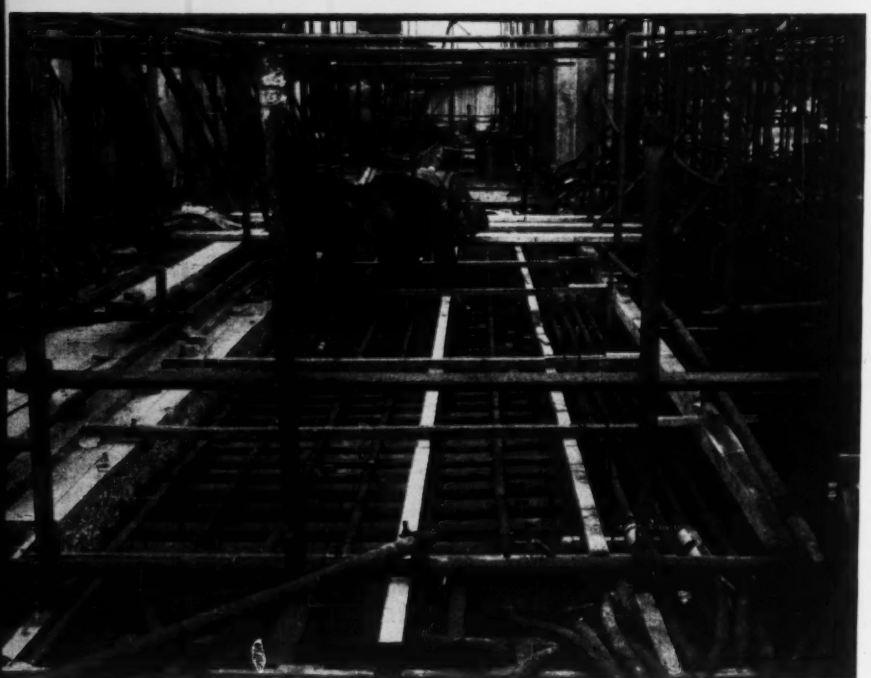
Since the bridge crosses the river at an angle of $31^1/_2$ deg, deep girders were used to give the least possible deflection on the side of the deck opposite to the forward bearings of each skew abutment.

As a consequence, the bridge is quite economical in prestressing steel. The total weight of high tensile steel bars used in the entire structure is only 28.4 tons. There are also 12 tons of mild steel reinforcement, mainly in stirrups and in the compression (top) boom, plus 20 tons in the abutments. The bearing plates of the 337 high-tensile-strength bars have a total weight of 3.65 tons.

Abutments are of reinforced concrete of the cellular counterfort type, built on pile foundations. The fixed end of the bridge is carried on rocker bearings, and the expansion end rests on two combined roller and rocker bearings, as shown in one of the photographs.

The deck slab gives more than normal side clearance. The girders are spaced 17 ft 6 in. on centers. The deck slab, which is only $10^1/_2$ in. thick, is prestressed transversely by $1^1/_8$-in.-dia alloy steel bars, on 9-in. centers, giving a stress distri-





Above Left:
Here ducts for floor and girder prestressing bars can be seen at end of bridge. Note dowel rods for casing end anchorages, and grout pipes for bonding prestressing bars.

Left:
Rubber tubes running both longitudinally and transversely are in place prior to concreting, to form ducts for high-tensile-strength prestressing bars.

Right:
On nearly completed bridge, workmen are applying prestress to bars which run transversely through floor. Gaps in girder have not yet been concreted in. About seven days after this was done, girders were being prestressed. Bridge is being placed in service in January 1953.

bution in the concrete at mid-span, after full creep and shrinkage losses, ranging from 1,795 psi in compression to 65 psi in tension in the unloaded slab. Because of the dead load of the track, the small tension is of course not realized. With full live load and impact the stresses are 148 and 1,610 psi in compression. The deck slab is prestressed longitudinally as well as transversely.

The main girders at midspan have a depth of 12 ft 6 in. Stress conditions under dead load after full losses are 517 psi in compression at the top and 1,620 in compression at the bottom. Under full load and impact these compressive stresses become 1,864 and 261 psi respectively. The maximum shear at the ends of each beam is 325 long tons, giving a normal maximum shear of 338 psi. The maximum principal stresses are 752 in compression or 152 psi in tension. The calculated upward deflection on application of prestress is 1.99 in., and the calculated deflection under full dead load, 1.24 in. A slight camber of 3 in. was given to the span; the railway track was also laid to this camber.

The concrete mix for the ordinary reinforced concrete of the abutments was 1:2:4, and the maximum slump 4 in. For the prestressed concrete, of which there were 333 cu yd, the mix was 112 lb of rapid hardening cement to 162 lb of sand, with coarse aggregate made up of 145 lb of $^3/_8$ in. down, and 235 lb of 1 in. down to $^3/_8$ in. The coarse aggregate was reasonably rounded. Practically all the aggregate is siliceous material.

The specified 28-day cube strength was 6,250 psi, corresponding to a cylinder strength of about 5,200 psi. Test-cube strengths varied between 7,000 and 8,000 psi, with occasionally higher results.

The water-cement ratio for the concrete in the deck slab and the compression booms of the girders was 0.38, increased to 0.42 at the bottom of the girders, where the complex arrangement of rubber tubes and secondary mild steel reinforcement made flow of the concrete more difficult, even under vibration. Compaction of the test cubes was by electric hammer; compaction of the bridge concrete was by a combination of one electric hammer on the formwork and two internal poker-type vibrators. The compaction of the placed concrete is believed to be at least equal to that of the test cubes.

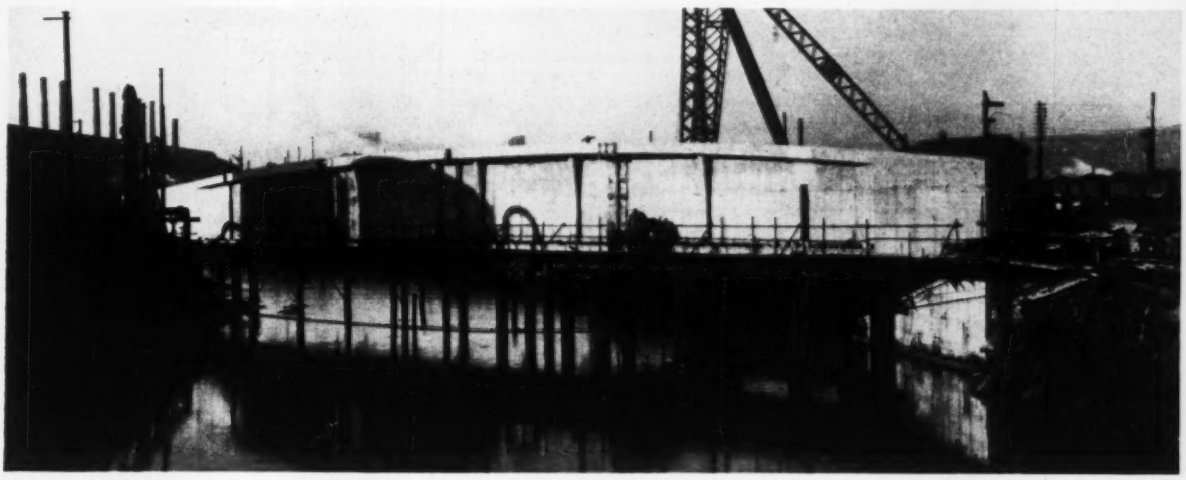
Density of test cubes was close to 153 lb per cu ft. Although I have not been able to find any precise relationship between strength and density, even for a particular site, an occasional drop in density is of course a sure sign of under compaction.

Prestressing bars, of high-tensile-strength alloy steel, totaled 337 in number. Ducts for these bars were formed partly with soft rubber tubes in which a loose bar was inserted, and partly with inflated rubber tubes. No trouble was experienced with the former but a few of the latter moved, it is assumed, during the internal vibration of the concrete, making it necessary to grind out the sides of some of the holes before the bars could be placed.

To make room for inserting the bar couplers, and to reduce shrinkage tension in the concrete before stressing, the girders and the deck slab were concreted in sections with gaps between. These gaps were concreted after the bars had been placed and the couplers correctly screwed on and wrapped to allow the necessary movement during stressing. As the bulk of the concrete had ample time to harden so that shrinkage and creep losses after stressing would be lower than normal, stressing of the girders was permitted seven days after the gaps had been concreted. After prestressing, the bars are being completely bonded to the concrete by pumping grout, composed of one part cement to one of sand, by volume, into the annular spaces between the bars and the holes.

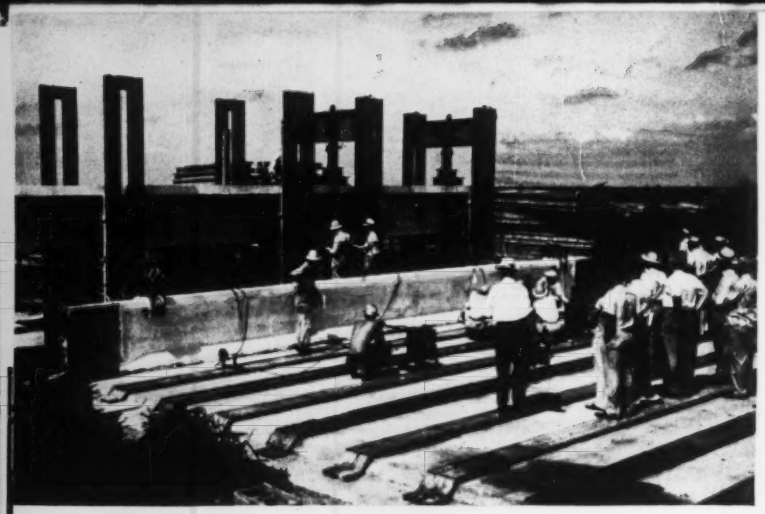
It would of course have been more economical to follow the more usual design in which the main girders are below the deck, if there had been sufficient room between the floor of the bridge and the high-water level in the river. However, the bridge is light considering the span and the loading, and apart from the fact that the prestressed concrete design was cheaper than steel or reinforced concrete, the very shallow construction depth saved a substantial amount by reducing the height to which the railway tracks had to be raised on both sides of the river.

The contractor was Messrs. George Longden & Co. of Sheffield. Work was begun in September 1951. Construction of the coffer-dams for the abutments, driving of foundation piling, and placing of abutment concrete took as long as the construction of the bridge span itself. The structure was designed by the writer as consulting engineer to Messrs. Steel, Peech & Tozer.

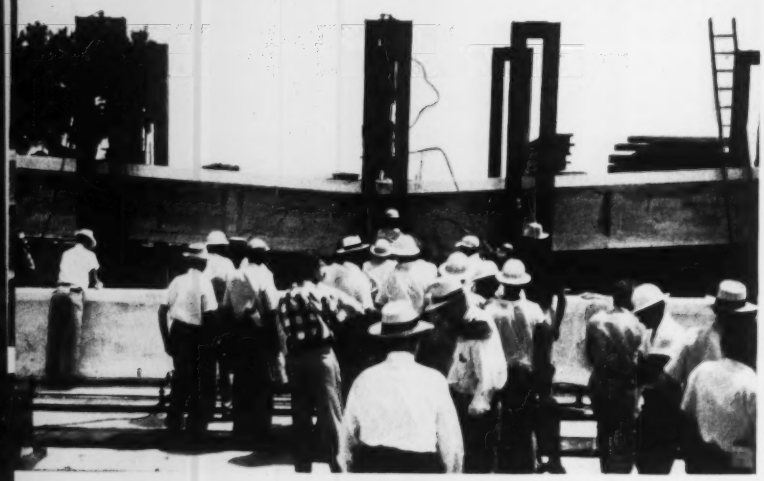


Tests establish

for prestressed



Prestressed composite beam, when tested for shear (top) up to H-60 loading, showed maximum deflection of 0.009 ft with no cracking or other evidence of distress. Jacks were then moved into position to apply bending load to same beam. At failure (lower view), beam was carrying load of about H-72.



The 15-mile crossing of Florida's Lower Tampa Bay, now under construction, will include approximately $3^1/_2$ miles of concrete trestle constructed with precast and prestressed concrete beams. This type of structure won its place in the project by virtue of being bid low in competition with two other more conventional trestle types. The net saving on the overall trestle cost amounted to approximately 4 percent.

This job is not only the first prestressed construction undertaken by the Florida Road Department, but also the first use of large high-strength bars for prestressing reinforcement in bridge members in this country. In overall length and in number of prestressed members, it is believed to be the largest installation of its kind in the world.

General information on the Lower Tampa Bay Bridge and the prestressed design in particular was given in the article by M. N. Quade, M. ASCE, in CIVIL ENGINEERING for April 1952 (p. 25).

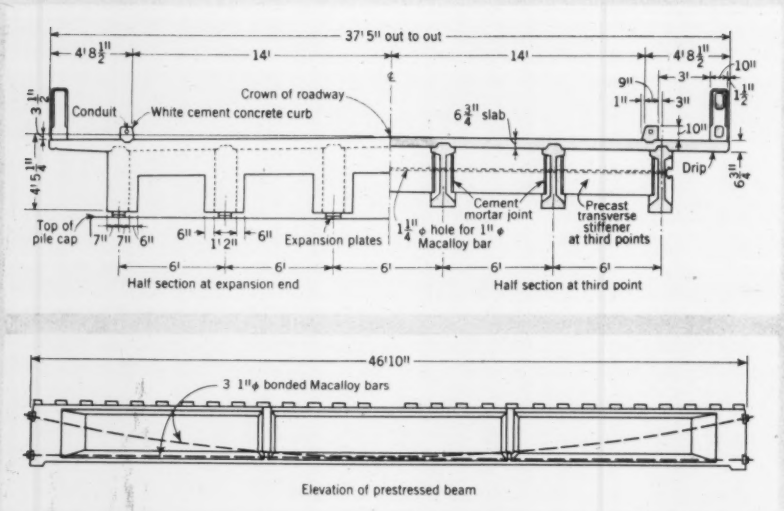


FIG. 1. Prestressed beams 48 ft. long are used for about 3½ miles of concrete trestle in Lower Tampa Bay crossing.

construction procedures

beams in Tampa Bay Bridge

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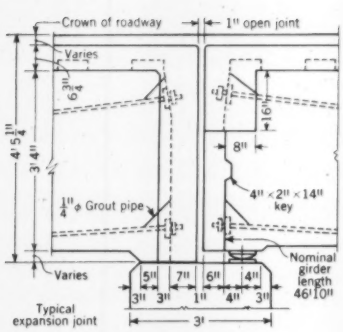
State Road Department

Tallahassee, Fla.

The winning design for the trestle sections, here discussed, consists of a total of 363 simple spans of 48-ft length each, supported by bents of 20-in.-sq reinforced concrete piles and carrying a roadway 28 ft wide with 3-ft walkways cantilevered on each side. Each span consists of six precast and prestressed concrete beams supporting a poured-in-place deck slab. End diaphragms between the beams will be poured in place, but intermediate diaphragms, at the span third points, will be precast, set and grouted in position and stressed normal to the center line of the bridge prior to placing of the deck slab.

The principal prestressed members in this design are the precast beams spanning between the pile bents. These are I-shaped, 3 ft 4 in. deep, with webs 4 in. thick and flanges 14 in. wide. At their ends they terminate in solid bearing blocks. These beams are proportioned to carry all the span dead load. The top flanges carry a system of concrete keys $2^1/_2$ in. high and projecting stirrups to insure action of the deck slab with the beam as a T-section under live loads. See Fig. 1. The beams are to be cast, cured and stressed in the contractor's yard and barged about 15 miles to the job site. After the beams have been erected and the intermediate diaphragms placed and stressed, the remaining construction of the deck slab will be quite similar to that for steel I-beam spans.



Details of Prestressed Design

The contract plans were developed by the Preload Company of New York and, after review and some modification, were approved by the Florida Road Department and its consulting engineers, Parsons, Brinckerhoff, Hall and Macdonald. The stressing system is based on the use of three 1-in. round, high-strength bars per beam. Two of these run nearly straight through the center of the bottom flange (Fig. 1). To eliminate tension in the top flange near the beam ends, the third bar is placed in a parabolic form curving upward from the bottom flange at mid-span to near the top of the beam at the ends. The precast intermediate diaphragms for use at the span third points will be stressed by a single bar through their centers.

In casting the members, open holes are left in the bar positions. Placing of bars and stressing are done after the members have been cured. To provide anchorage for the bars, their ends are threaded and fitted with patented, high-efficiency nuts. After a bar has been properly stressed, these nuts are turned up to bear against steel bearing plates, which transfer the force to the concrete.

Fluid grout is then forced through the holes to completely fill the annular space between the bar and the sides of the hole—a very important feature of the design. As initially stressed and anchored, the bar end anchorages must withstand all of the initially applied force. After grouting, subsequent changes of steel stress due to the application of live loads and impact are considered to be absorbed by the bond between the bar and the grout. By this means, the end anchorages are relieved of repeated variable stresses. Tests conducted on the bar and nut assemblies before the contract was awarded showed this condition to be a requisite for a safe design where heavy moving and impact loads have to be resisted.

To make holes for the stressing bars in the beams at the time of casting, the contractor proposed to install $1^5/_{16}$-in.-dia inflated rubber tubes in the forms, deflating and withdrawing them after casting. Of the three beams cast by this method, two had to be rejected on account of misalignment of the holes, and in the third beam the bars were only installed with some difficulty. It is obviously important to avoid friction between the bar and the sides of the hole in order to secure uniform stressing and proper elongation of the bar.

The rubber tubes have a wall thickness of $^3/_{16}$ in. and when inflated to an air pressure of 30 lb have remarkable stiffness. The contractor exercised great care in placing, but the very small tolerance in alignment which had to be maintained through a concrete pour 47 ft long, throughout vibration and removal of supports, presented a problem for which no immediate solution was evident.

The contractor then proposed the use of spirally wound flexible metal tubes $1^1/_2$ in. in outside diameter to form the holes for the prestressing bars. Under this procedure the tubes, with the bars inside them are installed in the forms before pouring of concrete. The tubes are tied firmly in position and their ends are sealed against the intrusion of concrete. A test beam was constructed

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by this method and the bars were found to be absolutely free within the tubes. Bar elongations checked with the jacking force exerted. This method was accordingly accepted, the contractor furnishing the metal ducts without extra compensation.

Grout Must Develop Bond

Grouting of the small annular space 47 ft long between the 1-in. bar and the side of the $1\frac{5}{16}$-in. hole is an unusual problem. Grout pipes $\frac{1}{4}$ in. in diameter are tapped into the holes at each end. The grout consists of 1 part portland cement, $\frac{1}{4}$ part fly ash, $\frac{3}{4}$ part screened and washed beach sand; with 4 to 6 gal of water and 1 lb of Plastiment used per bag of cement. The consistency is approximately that of heavy paint. Grout is applied continuously under moderate pressure through the pipe at one end until there is a steady exhaust through the pipe at the other end. The pressure is then raised to about 35 psi and the pipes closed under this pressure.

The grout must be capable of developing bond sufficient to absorb all variations in steel stress due to the passage of live loads. This bond requirement is very small. Under an H-20 live load and impact, the maximum at the bar ends is 8 psi. Several pullout tests have been made on specimens 3 ft long with holes formed both by the rubber cores and by the metal ducts. Fourteen-day pullout tests with both types of holes indicated bond strengths of about 200 psi. Grouting of these short specimens may have been conducted under more favorable conditions than would obtain for a long beam, but the high test values indicate an ample margin of safety.

In stressing the bars, care is taken to keep the stresses symmetrical about the vertical axis of the beam. Each of the two bottom bars, which are nearly straight, has its threads extended at only one end for attachment to the jack. One bar has its extended threads at one end of the beam and the other bar has the threads at the opposite end. Both bars are stressed at the same time by one jack at each end of the beam. The jacks are then moved up to exert stress on both ends of the curved upper bar, which has extended threads at both ends. The jacks are equipped with vernier scales for measuring bar elongations to 0.01 in. and with pressure gages for measuring the jacking force.

After the contractor had set up his plant for manufacturing the 2,178 beams required for the job, routine production was begun early in September and for the past four months has been carried on at the rate of 24 to 36 beams per week.

Concrete used in the beams is of high strength, to develop a minimum compressive strength of 3,600 psi at 7 days and 5,000 psi at 28 days. The concrete for the poured-in-place decks will be one of the Road Department's standard mixes with a minimum 28-day strength of 4,000 psi.

The specifications permit partial prestressing of the beams, sufficient for handling stresses, when the concrete has reached a compressive strength of 3,000 psi, and full prestressing at a compressive strength of 4,000 psi. A strength of 3,000 psi is being realized at an average age of five days. It is therefore possible to use the entire casting yard once a week. However, actual production is proceeding at about half this rate.

Because of the pioneering character of this construction, a rather elaborate program of testing full-size members was undertaken. The initial tests required by the specifications called for two beams. One was specified as a beam alone, constructed exactly like the beams in the bridge and hereafter referred to as the "bare beam." The other was a similar beam with a 6-ft-wide section of deck slab poured on it after it had been stressed and the bars grouted. This specimen will be referred to as the "composite beam."

The purpose of testing the composite beam was, of course, to study the action of the T-section consisting of beam and slab, which was considered in the design as resisting live loads.

The purpose of the bare-beam test was to establish an empirical standard of quality to apply to the manufacture of beams for the job. The bare beam was tested in bending by a gradually applied load at midspan. One in every 200 beams manufactured is being similarly tested. The specifications require that these beams shall not show a visible crack at a load less than 90 percent of that which produced the first visible crack in the initial bare beam.

Final tests will be made on two specimens identical with the bare beam and the composite beam to be selected at random from the beams manufactured near the start of the work and stored for a year, or until just before completion of the job. The purpose of these final tests is to evaluate any long-time loss of prestress by comparing the results with those of the original tests.

The initial tests on the composite and the bare beam were performed on

TABLE I. Tabulation of Design Stresses in Composite Prestressed Beam

Load Conditions	Concrete Stresses						Steel Stress	
	Before Losses			After Losses				
	Top Fiber		Bottom Fiber	Top Fiber		Bottom Fiber	Initial Stresses in Bars	Stresses Under Loads
	Girder	Slab		Girder	Slab			
(1) Prestressing	+511	....	−2,070	+441	....	−1,742	94,400	79,400
(2) Dead load of girder	−375	....	+ 348	−375	....	+ 348	....	....
Total, (1) + (2)	+136	....	−1,722	+ 66	....	−1,394	94,400	79,400
(3) Dead load of slab + stiffeners	−683	....	+ 586	−683	....	+ 586	....	3,000
Total dead load	−547	....	−1,136	−617	....	− 808	....	82,400
(4) H-15 (live load + impact)	− 99	−215	+ 583	− 99	−215	+ 583	....	3,200
Total, (1) + (2) + (3) + (4)	−646	−215	− 553	−716	−215	− 225	....	85,600
(5) H-20 (live load + impact)	−129	−280	+ 758	−129	−280	+ 758	....	4,150
Total, (1) + (2) + (3) + (5)	−676	−280	− 378	−746	−280	− 50	....	86,550

\+ Denotes tension
− Denotes compression

July 15, 1952, with the assistance of faculty members and senior students from the Civil Engineering Department of the University of Florida. A very thorough series of strain and deflection measurements were taken by University personnel. The more obvious results are reported here.

Tests on Composite T-Section

The tests of the composite beam to be used in the bridge are probably the most interesting. On the date of testing the ages of the various parts of the specimen were as follows:

Concrete in beam	63 days
Time elapsed since stressing	53 days
Time elapsed since grouting	50 days
Concrete in slab	45 days

Properties of the concrete at the time of testing as shown by the averages of a series of beams and cylinders which were field cured exactly like the composite beam and broken on the day of testing were:

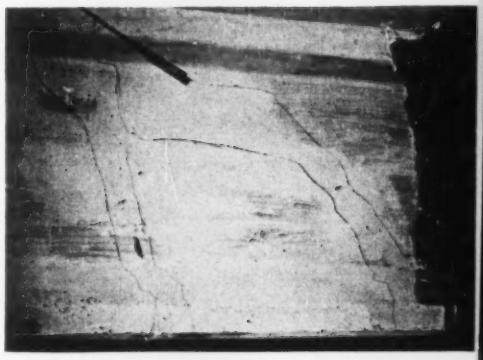
Concrete in beam:	
Compressive strength	5,500 psi
Modulus of rupture	635 psi
Modulus of elasticity	4.78×10^6 psi
Concrete in slab:	
Compressive strength	4,610 psi

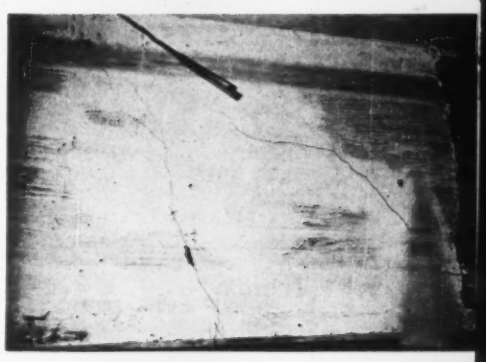
The composite beam was the one that had been acceptably cast using rubber tubes to form the holes for the rods. It had originally been intended to test it when the slab was 28 days old. However, the trouble and confusion incident to the casting of the two other rejected beams and the substitution of the flexible metal ducts delayed the manufacture of an acceptable bare beam, which was to be tested on the same day. This delay had the advantage of allowing more of the losses due to shrinkage and creep to take place. According to the best information available, considerably more than half of the eventual losses had taken place prior to testing.

The composite beam was required to meet definite specified criteria both in bending and in shear. These criteria were specified as multiples of standard AASHO highway H-loadings. As shown by the stress table, Table I, the beam was designed to maintain a small compression in the bottom fibers under H-20 live load. The specifications required that there be no visible cracking under a bending load equivalent to H-20 live load and that the section stand both a bending and a shear load equivalent to H-60 without failure. Failure was defined as permanent damage due to rupture of the steel or crushing of the concrete, or both.

A heavy reinforced concrete slab was constructed under the entire testing area, with concrete piers for beam supports constructed on top of the slab. Curved bearing plates 46 ft between centers were cast in the beam ends. These rested on steel bearing plates on the pier tops. The loading was applied by hydraulic jacks working between the top of the beam and steel yokes anchored to the underlying slab.

In applying the loads for both bending and shear, jacking was done from two points 14 ft apart, and the distribution of load between the two jacking points was maintained in the ratio of 1 to 4. The spacing and distribution of loads thus simulated the axle spacing and weight distribution of a standard H vehicle as specified by the AASHO. A simple multiplier applied to the load on the





In second set of bending tests on composite beam, load was applied in increments of 20 H up to H-60. From H-40 to H-60, cracks developed rapidly through middle half of span to maximum width of about $^1/_8$ in. When load was removed, all cracks closed, although deflection of 0.016 ft remained at midspan. Photos show cracks at right of beam center before (top) and after (below) removal of H-60 loading. Closing of cracks on removal of load is evident. In lower view, crayon has been used to outline bands in center of which hair-line cracks run.

TABLE II. Stresses in Composite Beam During Testing

Based on assumption that two-thirds of eventual loss of prestressing force had occurred at time of testing

Load Condition	Concrete Stresses: Top of Slab	Concrete Stresses: Bottom of Beam	Steel Stress
(1) Dead load and prestress	....	−916	87,400
(2) H-24 (live load + impact)	−336	+914	4,970
Total, (1) + (2)	−336	....	92,370
(3) H-40 (live load + impact) uncracked	−560	+1,516	8,300
Total, (1) + (3)	−560	+ 600	95,700
Total stress in cracked section under H-40 test load, with linear stress distribution in concrete, and load resisted by simply reinforced section.	−863	....	123,900
Total stress in cracked section under H-60 test load, with rectangular stress distribution in concrete	−2,720	....	151,800

\+ Denotes tension
− Denotes compression

Casting yard for 48-ft concrete beams consists of six concrete-floored pallets, each pallet having space for 12 beams. Side forms for beams are of $^3/_{16}$-in. steel plate backed by framework of structural steel shapes. Rail-mounted whirley crane handles concrete to forms and completed beams to storage areas and to barges.

jacks gave the H-rating directly. For the shear load, the heavy jack was placed within 2 ft of one end support, and for the bending load it was placed in the span center.

The shear test to H-60 loading was performed first, and to summarize, nothing of concern happened, except a maximum deflection of 0.009 ft near the span center. There was no cracking or other evidence of distress.

The jacks were then moved into position for applying the bending load. This load was applied in three increments up to the equivalent of H-20, causing only a mid-span deflection of 0.007 ft. The load was then increased in five increments to H-40, the large audience present inspecting the beam at each increment. Just as the H-40 load was reached, a slight pop was heard, and the first significant crack showed at mid span, extending completely through the bottom flange and to about one third the height of the web. Deflections had been increasing at an average rate of about 0.0025 ft per increment, but at the last increment, the deflection increased 0.008 ft. The total deflection at this loading was 0.025 ft.

The load was then increased to about H-45, principally to obtain a photograph, at which stage the crack had extended the full height of the web. On reduction of the load to H-20, the crack entirely disappeared. On complete removal of the load, the beam recovered its entire deflection. This was a little surprising, as the steel had probably undergone a slight permanent elongation. However, the effect of bond had evidently restricted the overstressed section to a short length.

The load was then applied in three increments up to H-60. Up to H-40 the structural behavior was not noticeably different from that under the first loading except that the crack became visible at about load H-25. At H-40 the deflection was again read as 0.025 ft. During the latter stage of the increment from H-40 to H-60, cracks began developing rapidly through the middle half of the span on 2- to 4-ft centers, and the deflection increased from 0.025 ft to 0.145 ft. The maximum width of cracks at this stage was about $^1/_8$ in.

The load was next completely released and the effect of permanent elongation of the steel was clearly evident. A deflection of 0.016 ft remained at mid span, and although all cracks closed, they remained clearly visible as fine hair lines.

Next the load was applied with the intention of breaking the beam. Up to H-60 the structural behavior was not different from that under the previous loading except that the cracks seemed to open slightly wider and there was an additional deflection of 0.035 ft. Between H-60 and H-70, the beam began to fail rapidly with yielding of the steel. Cracks opened up to as much as an inch in width. Large horizontal cracks along the bottom flange and diagonal cracks in the web opened, indicating failure in shear and diagonal tension. The top of the slab began to crush, and deflections increased at a greatly accelerated rate. Although the maximum load reached about H-77, excessive deflection from yielding of the bars was such that this load could not be maintained.

Actual collapse came under an applied load of about H-72 when in succession both bottom bars broke and the top slab crushed through its full depth. The last reading before collapse showed a total deflection of 1.86 ft.

One of the principal purposes of the composite beam test was to observe the action of the beam and slab as an integral T-section. The results seem to confirm completely the validity of our assumption of composite action. At no time during the tests was there the slightest evidence of differential slippage or separation between beam and slab. Eight strain-gage points on the slab and two on the upper flange of the beam indicated a distribution of stress over the entire section.

Stresses in Steel and Concrete

An approximation of the stresses in the steel and concrete during testing is perhaps of interest, but this is offered with some reservations.

Any computation of stresses must include some assumption of loss of original prestressing force due to creep and shrinkage. For design purposes the total eventual loss was taken as 15,000 psi in the steel. At an age of 53 days after stressing and 45 days after pouring of the slab, the greater part of the losses would probably have taken place. The losses were accordingly estimated at 10,000 psi and Table II is based on that assumption. Considering the action of the beam during testing, this estimate, is probably not far out of line, and the stresses shown for loads up to H-40 should be a close approximation.

After the beam was cracked and the load released and reapplied, the first crack became visible at about load H-25. It will be noted that the stress tabulation, Table II, shows zero stress in the bottom fiber of the concrete at H-24. On the first application of load where the beam cracked at H-40, the stresses just before and after cracking are shown. It will be noted that the bottom fiber stress before cracking was 600 psi, which compares fairly well with the 635 psi modulus of rupture of test specimens broken the same day.

The stresses shown for the H-60 load are certainly questionable, as the distribution of stress across the section and the actual location of the neutral axis are very uncertain. Based on the known average ultimate strength of the bars, the steel stress shown is probably high and the concrete stress correspondingly low. Yielding of the steel and consequent release of some of the prestressing force could explain this. The ultimate strength of the bars averages about 150,000 psi, whereas the steel stress for H-60 loading is shown as 151,800, and before breaking, the beam withstood a load about 25 percent higher.

Useful Conclusions Drawn

While the tests described here were limited in scope, the following tentative conclusions are perhaps justified for a project of this type:

1. The structural behavior of prestressed members can be predicted within a limit of accuracy comparing favorably with that of other structural types.
2. For heavy working loads and for some considerable overloads, the structure should remain crack free. For a concrete structure in a warm, humid atmosphere, constructed over salt water, this is a very important consideration.
3. For very severe overloads resulting in cracking, the cracks should close on removal of the load and the strength of the structure will not be impaired.
4. The tests demonstrated the rigidity which is a claimed characteristic of prestressed construction. Under any application of expected working loads the deflections were negligible. Under H-40 loading and after cracking of the beam, the deflection was only $^5/_{16}$ in. in a span of 46 ft, or $^1/_{1,800}$ of the span length.
5. The performance of the beam has fully complied with all specified criteria and this design for the Lower Tampa Bay Bridge is expected to result in a sound and serviceable structure.

(This article is based on the paper presented by Mr. Dean before a joint session of the ASCE Structural Division and the American Concrete Institute, presided over by A. E. Cummings, chairman of the ASCE Research Committee, at the Chicago Centennial Convention.)

What do we need to know about prestressed concrete?

N. M. NEWMARK, M. ASCE, Research Professor of Structural Engineering, University of Illinois, Urbana, Ill.

A great many problems in prestressed concrete face the conscientious designer. An obvious major difficulty is the fact that we have no design specification or code. Those who point this out most vigorously may be overlooking the more important fact that we have no basic philosophy of design to govern the preparation of such a code.

We know a good deal about many details, and we are rapidly acquiring additional knowledge from the research laboratory and from field experience. However, until we add engineering judgment and make the decision as to what we should attempt to design *for*, with regard to a code we are in the position of not being able to see the forest because of the multitude of trees in the way.

Although it seems entirely possible to draft a code for design in prestressed concrete in a form similar to that in use for ordinary reinforced concrete, the end results in terms of strength or useful limit loading may be entirely unsuitable. The reason for this situation is, of course, the not-too-well-recognized fact that design procedures and methods of analysis for reinforced concrete are essentially empirical rather than rational. This does not imply that such procedures are inadequate. It does mean that techniques which are empirical cannot be extended to other conditions or materials than those for which they were devised.

Although we are accumulating a good deal of experience with prestressed concrete, the interpretation of this experience in terms of strength or satisfactory performance is not a simple matter. Such field experience will, of course, indicate some of the conditions that are completely unsatisfactory, but it cannot measure possible undue conservatism in design. Moreover, the most severe conditions for which provision should be made in a design for a class of structures may not ever be encountered in a particular structure. The fact that a structure stands up does not always mean that it is satisfactorily and conservatively proportioned.

Our factors of safety in general are large enough so that ordinarily little information can be obtained from the performance of a structure unless it is unsatisfactory by an extremely wide margin, or unless we have some theoretical basis for interpreting the observations made on it.

Old Types at a Disadvantage

Well established methods, materials, and structural types always are at a disadvantage in competition with new ideas. It might appear that lack of experience with new procedures would cause them to be treated with more conservatism than old ones. However, this has not generally been the case. It is an easy matter to reduce factors of safety when one does not know what they are, in order to make the new structure appear economical as compared with the old. This reduction may not be serious. We all realize that it would be possible to increase working stresses or reduce safety factors for commonly accepted structural types without running into serious difficulties.

In fact, the more general and widespread is a particular type of structure, the more conservative are the codes that provide for its design. This point can be best illustrated by reference to the design procedures for slab floors in buildings. The advent of the so-called flat-slab type of construction saw the introduction of designs which involved tremendous savings in materials and costs over more commonly accepted competing structures. As rational methods for the analysis of slab structures were developed, the design provisions became more conservative. Yet the advantages given to flat-slab construction, in terms of code design provisions, exist to the present day.

The lack of a basic design philosophy is now, and has always been a cause of difficulty in the evaluation of new concepts, materials, and techniques. The use of prestressed concrete, although it possesses advantages which are real and important, may either be benefited or hurt until a basis for the evaluation of this new material relative to ordinary concrete is established. Whether the comparison be in terms of ultimate strength, working loads, or some modified combination of the two, the decision as to the choice of a factor of safety will be the primary and perhaps the only important criterion in the choice between these two methods of construction in reinforced concrete.

In arriving at the proper solution of the problem, the role of research is most important. In this respect, research has three aspects: (1) analysis and analytical study, (2) laboratory experiment, and (3) observation and experience. The analysis is required to tell us what to look for and what to test; the laboratory research permits us to evaluate different ranges in behavior up to and including ultimate strength; and field experience and observation delineates the practical problems of construction and design that must be taken into account.

The writer feels that concentration of effort applied on these various directions will lead us shortly to a satisfactory basis for the evaluation of prestressed concrete and for the development of proper regulations for design. It is to be hoped that in the process of solving this particular problem, the entire problem of engineering design will have been put on a firmer basis so that future similar problems can be more readily resolved.

Three fundamental concepts considered

1. Working Loads, Useful Strength, and Ultimate Strength

Although there may be some ambiguity in the definition of "normal" conditions, it is fairly convenient to consider that these are the conditions under which a structure is subjected to its usual, normal, or design load. Of course, somewhat improbable combinations of loading may have to be combined for this condition. However, if we use a "working-load" specification, the stress at this load is arbitrarily but unequivocally defined. If we use a load factor or ultimate load specification, the stress at working load is less definite but the uncertainty in this magnitude does not cause us undue difficulty.

There is always, and there always should be, a considerable margin between the working load and the useful strength of the structure. The useful strength is defined as that load or combination of loads beyond which some undesirable action of the structure occurs. This may be too large a deflection, too great a stress, too much cracking, or some other condition which impairs the appearance, the capacity, or the deformation of the structure.

There is less ambiguity and uncertainty about the ultimate strength, which is, of course, measured by that load or combination of loads which produces failure or collapse of the structure.

In any comparison of prestressed and ordinary concrete we may have different criteria for the different conditions. At working loads, it is considered permissible to have some cracking in ordinary concrete but it has been uncommon until recently to permit cracking in prestressed concrete. Moreover, because of the difference in the materials, or possibly only because of the condition of prestress, there may be a difference in the type of failure of structures designed for the same condition in prestressed and in ordinary concrete.

2. Brittle and Ductile Failures

Although designers in structural steel have only recently become concerned with brittle failures, the designer in reinforced concrete has been aware of the distinction between brittle and ductile failure for a long while. As a matter of fact, building code specifications are written in such a way as to avoid brittle fractures by limitations on the stresses permitted in the concrete and the steel, and in other ways.

The same types of phenomena occur in prestressed concrete as in simple reinforced concrete. For under-reinforced beams the prestress is overcome before failure takes place. Such a failure is a ductile failure and generally is similar to that which would occur if the structure did not have initial prestress. However, in an over-reinforced beam, failure occurs by crushing of the concrete, and although the failure may depend on the amount of prestress, in general the steel does not yield before crushing of the concrete takes place. It would be entirely logical to provide for a greater factor of safety in an over-reinforced prestressed beam than in an under-reinforced beam because of the difference in the characteristics of the failure of these two types of member.

In our discussion of flexural failure we did not include other types of failure which are also generally undesirable and have the characteristics of brittle failures. These are failures in shear or diagonal tension (generally even more sudden and disastrous than failures in compression) and failures in bond, which may also be of the same type. Although failures in shear are less common in prestressed concrete, they can occur. Fortunately, provisions can be made to avoid them. Failures in bond or anchorage must be considered, and steps must be taken to avoid such failures in prestressed concrete.

3. Choice of Factor of Safety

However the engineer goes about designing a structure, he must either explicitly or implicitly choose a factor of safety or margin of safety of some sort. The mere selection of design loads, and the choice even of a method of analysis, sets a value for this factor. It is not my intention here to go into such matters as the choice of working loads or of maximum loads from the point of view of probability, and the influence of variations in material and other uncertainties on the necessary allowance to be made for material strength. These are certainly items to be considered but they are minor details compared with the fundamental philosophical problem, which is essentially a matter of engineering judgment in the selection of a factor of safety.

We have already discussed the reasonableness of allowing a greater factor of safety for undesirable types of failure. We must recognize that it is impossible to require all structures to be designed so that they fail in a ductile fashion. Such a provision may be grossly uneconomical in particular cases. The numerical differences in factor of safety for the different types of failure might perhaps be rationalized on the basis that these various failures are associated with a more nearly constant factor of safety based on energy absorption.

However, a major problem still remains. This is the selection of a factor of safety in terms of useful strength or in terms of ultimate strength, or the weighing of the factor of safety in terms of the margin between working load and useful strength, or between useful strength and ultimate strength. In high-strength steel, which is used for reinforcement in prestressed concrete, there is no definite yield point. Therefore there may be a considerable margin between the development of the yield strength and the ultimate strength of the steel. In this range, secondary failures may be developed.

How much allowance should be made in factor of safety for the range between the load at which cracking takes place and that at which initial yielding occurs? Should prestressed concrete be designed on the basis of a load factor, for either the maximum useful load or the ultimate load? How much allowance should be made for the range between these two values in selecting the factor? These points require an answer, even though it be an evasive one, before we can make either economic or adequate use of prestressed concrete. It is in this field that research must be concentrated—not only laboratory research but also the application of engineering judgment. Perhaps some further light on these points will be obtained in a comparison of the properties of prestressed concrete with ordinary concrete.

Prestressed and ordinary reinforced concrete compared

1. Relative Strength and Deflection

In general an ordinary concrete beam in the stage below cracking shows a linear relationship between load and deflection corresponding to the action of a homogeneous section. At the cracking load, the load deflection relationship begins to curve off from the initial slope and approaches another straight line which corresponds to the properties of the completely cracked section. The curvature, however, increases as the steel yields, and the load deflection curve becomes practically horizontal at the maximum load.

In a prestressed beam *of the same materials* the initial slope corresponding to the homogeneous section is the same, and the curve follows this line until the concrete cracks. The cracking occurs at a considerably higher load because of the action of the prestress. The curve then slopes over more sharply to reach about the same ultimate load and the same ultimate deflection as in the beam without prestress, for an under-reinforced beam. In other words, for moderate amounts of reinforcement the effect of the prestress on the maximum load-carrying capacity and on the maximum deflection is negligible. However, the shape of the load deflection curve is considerably different from that for a beam without prestress, and in general the useful energy absorbing capacity of the prestressed beam is higher. Even in the case where loads may be applied and released, where the level of loading is somewhat below the ultimate strength, the characteristics of the two types of beams are such that the ordinary beam has a much greater flexibility then the prestressed beam in the range up to and slightly above working loads, but the maximum deflection and maximum load are roughly the same for beams of the same materials.

Although the strengths of under-reinforced beams are not appreciably affected by prestress, it seems logical to permit a lower factor of safety in the prestressed beam than in the one without prestress because of the more favorable action at working loads and the greater energy absorbing capacity up to the maximum useful load. Just how much this differential should be is a question that must still be studied.

The effect of the prestress is not the only matter to be considered. There is in addition the effect of the difference in properties of the steel and concrete used in prestressed construction and in ordinary reinforced concrete. There is a further question also of the desirable amount of prestress. The shape of the load deflection curve can be controlled within certain limits by the magnitude of the prestress, and there seems to be no reason why the most favorable type of curve should not be provided.

For over-reinforced beams, the effect of the initial tension in the steel on the maximum load may be considerable. In general, the higher the initial tension the greater the capacity of the beam, provided that damaging effects are not produced in the unloaded beam by the large prestress.

Although it is generally accepted that compressive failures are to be avoided in reinforced concrete, they may not be as serious in a prestressed beam as in one without prestress. This is a point that should be studied further in terms of the relative economy of under-reinforced and over-reinforced beams (which is probably a function of relative costs of steel and concrete and the relative advantage of savings in dead weight). The writer would probably base his decision on relative energy absorbing capacity, but he must confess that considerable substantiating evidence would be required to induce him to prefer an over-reinforced to an under-reinforced prestressed beam.

2. Shear, Diagonal Tension, and Bond Failure

Even with no explicit provision for shear or diagonal tension, a prestressed beam has a greater resistance to shear than an ordinary concrete beam because of : (1) the additional compression in the concrete, which reduces the diagonal tension arising from the shearing forces; (2) the different nature of the diagonal tension failure which arises from the absence of a flexural tension crack to start this failure.

In a beam without diagonal web reinforcement or vertical stirrups, after a tension crack starts it tends to curve off into a diagonal tension crack under the influence of shear. When such a crack reaches the compression area of the beam, the shearing forces must be carried completely by the concrete section above the crack. In regions of high shear a tendency to induce a compression failure because of the combined stresses has been noted. This tendency is delayed in a prestressed beam because of the delay in the initiation of the diagonal crack to trigger the "shear" failure.

It is a relatively simple matter in under-reinforced beams to avoid shear or diagonal tension failures. In such beams these failures are as serious in prestressed as in ordinary concrete and should be avoided at all costs either by bending up reinforcement or by stirrups if the strength of the concrete alone is not sufficient.

There is some reason to believe from a few tests that have been made that shear failure in a prestressed beam without web reinforcement may be even more sudden than in a beam without prestress. This arises from the fact that the action of the prestress may be to delay the inception of a shear failure. However, once the failure begins, the influence of the prestress in ameliorating the conditions is no longer felt, and there may even be an excess of energy tending to separate the beam along a diagonal tension crack which forms suddenly rather than gradually. Therefore, unless the prestress is high enough to prevent the shear failure altogether, it does not seem wise to depend on it to avoid providing otherwise for shear and diagonal tension.

3. Bond and Anchorage

In prestressed concrete, as distinct from ordinary concrete, it is possible to depend entirely on the end anchorage of the reinforcement rather than on the bond throughout the length of the bar. On the other hand, such dependence introduces problems which have not been completely studied. In a prestressed beam, unbonded compared with bonded reinforcement generally involves a lower ultimate load. There may be some advantages from the point of view of construction operations in unbonded reinforcement. There may be corresponding disadvantages in fatigue

and creep in the anchorage details. A good deal of study is required to define the problem in such a way that it can be solved.

Because end anchorages may be needed to apply the prestress in the first place, the additional bond strength required under normal loading conditions is considerably less in a prestressed beam than in an ordinary beam. This is fortunate because of the difficulty in providing the proper bond resistance in a prestressed beam. In most instances the bonding of the reinforcement is accomplished by grouting after the steel is tensioned. The grout must be placed in such a way that its subsequent shrinkage does not destroy the bond resistance. This has been accomplished by the use of admixtures such as aluminum powder in the grout. Other ways may be more desirable.

Because of the different nature of the surface of high-strength steel as compared with the surface of ordinary structural steel, with which we are familiar, and because of the high stresses in the reinforcement in prestressed concrete, the subject of bond strength must be studied almost completely anew. Our background of experience in bond resistance, which was almost completely invalidated by the advent of the new type of deformed reinforcing bar, is again of little use in the problems of bond which arise in prestressed concrete.

In this regard it must be pointed out that we need only insure the existence of a sufficient bond strength to carry the changes in stress in the reinforcement due to the loading. This is a somewhat different condition from that required to develop the strength of a bar near its end. Of course, the conditions become more nearly similar in the case of prestressed and ordinary concrete when the problem is that of anchoring the bar by means of bond alone, as is the case in those uses of pre-tensioned reinforcement where the anchorage does not become a part of the final structure. Here experience has already indicated that unusually long lengths of embedment are required to develop the anchorage forces, and there is a real danger of progressive slip from the end of the bar to the middle.

Special problems discussed

The major problem in prestressed concrete is the fundamental one of selecting the proper basis for design, as previously discussed. Although this is a question which will require the most effort for its solution and for which even the technique for arriving at a solution has not been clearly determined, there are a number of specific problems which must also be solved in order to permit the most efficient and economical use of prestressed concrete. Seven of these problems are considered in succeeding paragraphs.

1. Bond, Anchorage, and Prestressing Methods

Some of the problems concerned with bond and anchorage have already been dealt with. Many of these problems are related to the technique used in applying the pre-tension or post-tension. Because many of the available procedures are patented, objective research has been somewhat difficult. Laboratories are rightfully reluctant to make comparisons among patented methods. It is to be hoped that sufficient fundamental research can be accomplished to permit the development of a number of procedures which will be satisfactory and that the choice among these procedures can be based primarily on their cost or on the cost of the equipment and special fittings required.

Much more development will be accomplished in the methods of construction, particularly in provisions for continuous structures and in the fitting together of precast units.

A recent development that seems to offer considerable attractiveness is the possibility of using bars instead of wires. Ultimately the choice among the various kinds of reinforcement and the various methods of fabricating prestressed concrete will be based on the simplicity of the construction methods and the cost.

2. Fatigue Strength

The fatigue strength of prestressed concrete appears to be almost entirely related to the bond strength under repeated loading and the strength of the anchorage under such loading. Of course, the fluctuation in stress in the reinforcement or at the anchorage is relatively small for the majority of the loadings that occur. It is well known that the stress in the reinforcement is practically independent of the load up to the load at which the initial tension force is overcome. However, even the slight fluctuation in stress that occurs under live loading is at a relatively high level, and the clamping or anchoring of the ends of the reinforcement inevitably produces stress concentrations. Tests of the adequacy of anchorages under even these small fluctuations should be made in those cases where it is proposed to depend entirely on the anchorages to develop the stress in the reinforcement.

3. Fire Resistance

The fire resistance of prestressed concrete has long been a subject of discussion. The major problem here, of course, is in the possibility of loss in prestress due to the changes in length or changes in physical properties caused by the heat. The insulating effect of the concrete should be taken into account. However, a secondary factor must be considered. This is the possible change in strength of the anchorage as it is heated and while it is subjected to sustained loading during the fire. A failure of the anchorage may require the load to be carried by bond, which may or may not be adequate under this severe temperature condition, or even under ordinary conditions.

4. Sustained Loading

The problem of sustained loading is perhaps of greatest importance in connection with the anchorage strength. However, there may be problems of creep associated with long-time live loading of high intensity. The creep may occur in bond along the bar, or the end anchorage may relax sufficiently to reduce the prestress. The stress levels which it is desirable to use in the reinforcement and in the concrete are also determined in part by the level at which creep begins to appreciably reduce the prestress force.

5. Reversed Loading

Prestressed concrete is particularly efficient for the resistance of forces which always act in the same direction. There would be little reason to consider prestressed concrete if concrete were as strong in tension as it is in compression, although even here the possibility of a

more economical structure with tension carried by high-strength steel might be inviting. However, when loads are reversed in direction so that the stresses may change in sign, then the problem becomes complicated by the fact that we must have our cake and eat it too.

For example, in a beam if the reinforcement in the lower side is prestressed in tension, the concrete on that side is prestressed in compression. Then, although the beam is favorably preloaded to resist positive bending moment, it is unfavorably loaded to resist negative bending moment. To prestress the reinforcement in both faces of the beam would increase the prospect of a compression failure in the concrete. All the steel might be placed at the center of the beam in an attempt to provide for loading in both directions, but this is efficient for neither direction. The basic problem still remains, "How can we best provide for reversed loading in prestressed construction?"

6. Blast Loading

This same problem of reversed loading arises when a structure is subjected to blast. In general, explosive forces may act in either direction on a structure, depending on where the detonation takes place with regard to the structural element involved. The provision for equal and opposite maximum loads seems to be one which is not easily met in prestressed concrete. However, there may be a solution to this problem which has not yet been considered. Further studies seem warranted because of the growing importance of blast loading in design.

7. Continuity

The problem of designing prestressed concrete for continuity has been discussed by A. L. Parme and G. H. Paris ("Analysis of Continuous Prestressed Concrete Structures," *Proceedings* of the First United States Conference on Prestressed Concrete, pages 195–206). The general problem is concerned with the influence of the prestress on the statically indeterminate quantities in the structure. In order to be consistent with concepts of ultimate strength, considerable further work is required to take into account the conditions which arise as the structure approaches failure or as plastic hinges develop in it.

So far as the writer knows, this problem has not yet been considered, although it is a basic one. It is no more reasonable to design a continuous structure on the basis of elastic behavior than it is to design a statically determinate beam on such a basis. Design for elastic action in prestressed concrete seems even more inadequate than such a design would be for ordinary concrete.

An attempt has been made to cover the most important points of uncertainty in our knowledge of prestressed concrete. These are the points concerning which research needs to be carried out, both in the laboratory and in the field. There are many quantitative problems of detail which have not been discussed, but the major problems of principle have been stated. It is to be hoped that attention devoted to these problems will permit us to make the best use of this new method of construction.

The most important aspect of the problem is concerned with the basic philosophy of design. This involves the selection of factors of safety or load factors, and must be solved before a consistent design procedure can be formulated.

In the preparation of this material the writer has had the benefit of discussions with his associates and colleagues, and with a number of members of the Joint Committee on Prestressed Concrete. Particular acknowledgment is due Professors C. P. Siess and I. M. Viest, and Messrs. J. H. Appleton and E. M. Zweyer of the staff of the University of Illinois.

(This article is based on the paper Mr. Newmark presented before a joint session of the American Concrete Institute and the ASCE Structural Division, presided over by A. E. Cummings, chairman of the ASCE Research Committee, at the Centennial of Engineering Convention in Chicago.)

How would you do it?

Some of the most fascinating chapters in the life and memory of an engineer are those which deal with the unusual and unexpected situations which almost got him down but from which he finally emerged the victor.—G. H. Gilkey

In the construction of the Pan-American Highway along the Pacific shore in Peru, engineers elected to locate the highway along the side of the sand mountains which slope into the ocean at an angle of approximately 60 deg to the horizontal. The sand is without moisture as the rainfall averages only $^1/_2$ in. per year. The highway runs about 100 ft above the ocean and 500 or 600 ft below the crest of the mountains or dunes.

It is assumed that the original section was made with an angledozer and that the sand was rolled into the ocean until a path of sufficient width for construction of the highway section was developed. At any rate the highway was constructed and immediately the problem of sand spilling on the roadway developed. In recent years, the highway authorities built rock walls about 6 ft 7 in. high, for a distance of about twelve miles, to hold back the sand. After a period of time, however, the sand began to pour over the wall, making it necessary to remove the sand constantly. It will block the highway within twelve hours in some places. A contractor has a full-time contract as sand mover.

Proposals have been advanced for a tunnel through the mountains or the building of a rock section along the route. Both of these ideas are extremely expensive considering that the annual highway budget is only about 12 million dollars.

A plan that I proposed may be found on page 71, but as far as I know it has not been tried. How would you do it?

EDITOR'S NOTE: This is the ninth installment of a series which started in the February 1952 issue of CIVIL ENGINEERING. In the April issue an article, "The Unexpected in Engineering: The Bugs," explains the project and enlarges upon the central theme that the problems of the past created the practice of the present; that "The engineering of today rests upon a coral reef: sturdy remnants of yesterday's bugs." The process is a continuing one; there will always be today's and tomorrow's bugs to add zest and gray hairs to the practice of a profession that in its very nature must cantilever from a codified past to an untried future. "Long live bugs" is an ever-present challenge to the virility and ingenuity of the engineer. If you have a good bug, why not share it? H.J.G.

The above problem was submitted by HENRY COOK, A.M. ASCE, consulting engineer, Guayaquil, Ecuador.

Weir plate holder is installed in end of sample piece of tile. Extra plastic weir plates are pictured at right.

ENGINEERS' NOTEBOOK

Weir plates helpful in sewer gaging

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In connection with the sewer engineering work of The Metropolitan District, Hartford, Conn., it has been necessary to devise sewer gages for two purposes:

1. To check compliance with specified requirements of the leakage rate in new sewers
2. To measure maximum water levels in overloaded sewers

It is frequently desirable to measure the infiltration or leakage of ground water into a newly built sewer, or other small flow.

The method previously used for measuring rate of leakage flow in new sewers involved timing with a stop watch the filling of 1- to 2-gal containers, after a mason had been employed to build a brick weir in a manhole invert with a pipe spout high enough to accommodate the container under the spout. With this method, a long wait was required for the leakage flow to fill the pipe and rise high enough to flow through the spout at a steady or uniform rate.

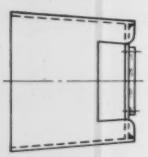


FIG. 1. Plan, section, and elevation diagrams give details of steel weir plate holder with Plexiglas weir plate bolted in place.

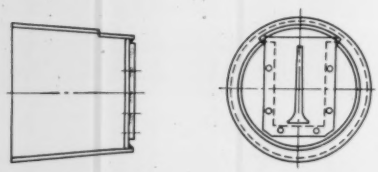
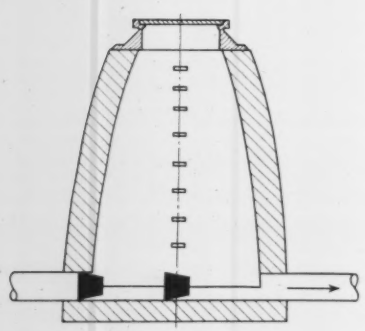


FIG. 2. Longitudinal section shows two typical positions of weir plate holder in manhole invert: (1) inserted in end of pipe at upstream side of manhole; and (2) set in manhole invert. Plate holder is calked with jute and clay to prevent leakage past outside of holder.



In an effort to simplify and shorten the leakage measurement procedure, a search for a better method was started. To measure the relatively small leakage flow, a V-notch weir was first considered, but rejected because of the great sensitivity of measurement of water level needed. To reduce the sensitivity, it was decided to try a Sutro proportional weir. Computations using the formulas (*Engineering News-Record*, November 12, 1936, page 679):

$$x/b = 1 - 2/\pi \text{ arc } \tan\sqrt{y/a}$$

$$Q = ca^{1/2}\, b\sqrt{2g(h - a/3)}$$

gave weir-opening sizes and shapes (Figs. 1 and 2 and photograph).

The openings were cut in the $^1/_8$-in.-thick transparent plastic (Plexiglas) which formed the weir plates. Then a weir plate holder of $^3/_{32}$-in. steel was devised in the form of a tapered cylinder to be calked into an 8-in. tile pipe at manholes with jute or clay. A rectangular opening with the necessary bolt holes was provided in the outlet or weir end of the cylinder to which weir plates of various shapes and flow capacity could be bolted. A small hand opening was left in the top of the weir plate end of the holder to permit the removal of any debris or foreign matter which might collect in the weir opening and interfere with its operation.

After the opening was cut, the weir plates were calibrated with a water meter and with timed filling of containers of known volume. The actual flows did not agree exactly with the computed flows for some reason—perhaps because the plate was thicker in relation to the area of opening than were the larger Sutro weirs used by Camp and others for measuring larger flows. The flow rates as measured were scribed on each weir plate.

To measure very low leakage rates, one plate was cut with a rectangular opening $^1/_{16}$ in. wide by $4^1/_2$ in. high. This was also calibrated, but of course did not produce variation in water levels proportional to the flow. With the three weir plates which were made up, it was possible to measure from 100 to 30,000 gal per day. This is equivalent to The Metropolitan District's maximum allowed leakage rate of 20,000 gal per mile per day for an 8 to 27-in.-dia sewer from 0.005 mile to 1.5 miles in length.

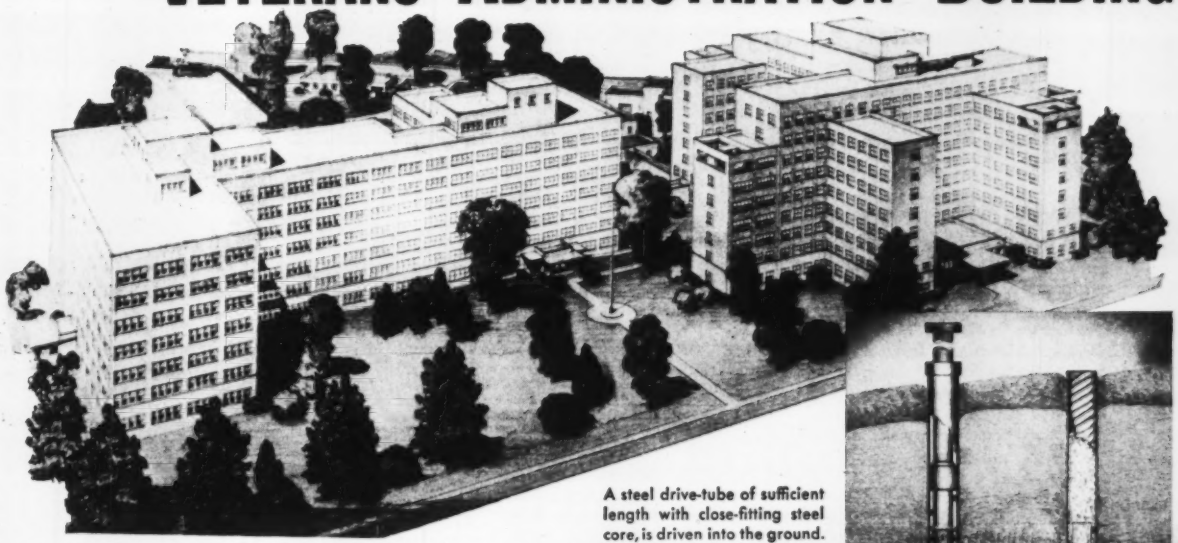
Because of the small ($^1/_8$-in.) clearance allowed, it was found that our weir arrangement would not fit inside some tile pipes which appeared to be out of round. As a result, we tried calking the weir with clay midway in the open half-pipe channel through the manhole. This position worked so successfully that it has been continued when necessary.

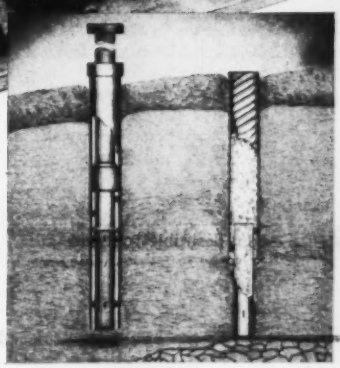
This weir did not raise the water upstream as high as the older brick weir and overflow pipe method. Installation of the weir was so simple that a skilled mason was not needed. Much less time and consequent cost were involved in installing and waiting for the flow to rise and arrive at a uniform rate. Direct readings of flow rate in gallons per day could be made by anyone and easily converted into gallons per mile per day. In fact, the general success of the proportional weir plates for small flow measurements prompted the writing of this article for whatever use the idea might be to others.

Credit for adapting these ideas to our use and setting up the equipment rests with fellow engineers and survey men in the Hartford Metropolitan District, Bureau of Public Works.

WESTERN *licks "Chicago Clay"*

DRIVES SECURE FOUNDATIONS FOR VETERANS ADMINISTRATION BUILDING





A steel drive-tube of sufficient length with close-fitting steel core, is driven into the ground. The core is removed and the closed end pipe section is driven out of the tube using the core as a follower. Foreign matter is excluded from the drive-tube by a gasket attached to the upper end of the pipe. When the driving of the pipe has been completed a corrugated shell is screwed down over the dogs (or anchors). These anchors are welded onto the pipe and develop a strong mechanical bond between the two sections. When required by site conditions, an alternate water-tight slip-joint is used. Both sections are then filled with concrete.

The subsoil at the site of the Regional Office and Clinic Building of the Veterans Administration in Chicago, is "Chicago Clay" at its worst. In view of previous foundation difficulties in this area, piling specifications placed full responsibility on the foundation contractor, while allowing a wide choice as to pile type and procedure. Western's Projectile pile was selected.

It was necessary to obtain approval of the type and method of installation. In accordance with the specifications, two groups of seven Projectile type piles each were driven and one pile in each group was subjected to an 80-ton load test. Measurement of tested piles showed less than 0.3 inch settlement under 80 tons. Specifications permitted 0.6 inch. This testing operation required 12 days.

Upon approval from the Veterans Administration, driving of the contract piling was started. Installation of 1380 piles was completed in 45 working days.

As the job progressed, five additional load tests to 80 tons each were conducted on piles selected by the Veterans Administration Resident Engineer. All were similar in behavior, with a net settlement of less than one-half of the allowable. No measurable horizontal movement of individual piles or groups of piles was detected. Although all piles were redriven, as required by the Specifications, practically no movement occurred, indicating that the slight heave which had been observed in the lower section, was due to compaction of the hardpan under the bottom of the pile.

Consultation is invited. Write for our catalog.

COMPLETE FOUNDATIONS FOR

INDUSTRIAL PLANTS, POWER PLANTS, PIERS AND DOCKS, BRIDGES, AIRPORTS, STEEL PLANTS, HOUSING PROJECTS, COMMERCIAL BUILDINGS, SCHOOLS, ETC.

WESTERN FOUNDATION CORPORATION

308 W. Washington St., Chicago 6, Ill. • 2 Park Avenue, New York 16, N. Y.

SAN FRANCISCO CONVENTION

SPONSORED BY THE SAN FRANCISCO SECTION,

TOGETHER WITH THE HAWAII AND SACRAMENTO SECTIONS

Headquarters: Fairmont Hotel San Francisco, Calif. March 3–6, 1953

REGISTRATION

FAIRMONT HOTEL. Main lobby. Hours: Monday, March 2, 9:00 a.m. to 5:00 p.m.; Tuesday, March 3, and Wednesday, March 4, 9:00 a.m. to 9:00 p.m.; Thursday, March 5, and Friday, March 6, 9:00 a.m. to 5:00 p.m.

Registration fee, except ladies and students, $5.00. Advance registration desirable to assure admittance to some functions which are limited as to numbers.

LOCAL SECTIONS CONFERENCE

Monday, March 2
Tuesday, March 3

9:30 A.M.
FAIRMONT HOTEL

Representatives of Local Sections of ASCE in the West will convene to discuss the expanding activities of ASCE at the Local Section level. The Conference, which is primarily for invited delegates of selected Sections, will be open to any and all who may be interested in the activities and operational details of ASCE Local Sections.

TUESDAY AFTERNOON MARCH 3

Air Transport Division

GARDEN ROOM
FAIRMONT HOTEL

Presiding: Terry Owens, Chairman, Executive Committee, Air Transport Division

2:00 Jet Transport Economics—Influence on Airports and Airways

JOHN G. BORGER, Chief Project Engineer, Pan American World Airways.

Discussion: RAY D. KELLY, Superintendent, Technical Development, United Air Lines, Inc.

3:00 Criteria for Planning Utilization of Space for Major Air Terminal

GEORGE D. BURR, M. ASCE, Senior Engineer, San Francisco Public Utilities Commission.

Discussion: CARL HAND, District Airport Engineer, Civil Aeronautics Administration.

Construction Division

TERRACE ROOM
FAIRMONT HOTEL

Presiding: A. H. Ayers, Member, Executive Committee, Construction Division

2:00 PANEL DISCUSSION: Effect on Construction Industry in Event of Reduction in Defense Spending

Moderator: L. B. Combs, M. ASCE, Rear Admiral, CEC, USN Ret., Head, Department of Civil Engineering, Rensselaer Polytechnic Institute

Speakers on following phases: Economics, Construction, Government, Financial.

Engineering Mechanics Division

VANDERBILT ROOM
FAIRMONT HOTEL

Presiding: Douglas McHenry, Member, Executive Committee, Engineering Mechanics Division

2:00 Investigations in Limit Design

JOSEPH S. DORFMAN, Lt., CEC USNR

2:45 Lateral Buckling of Aluminum Alloy Channels and Z-beams Subjected to Pure Bending

HARRY N. HILL, M. ASCE, Research Engineer, Aluminum Co. of America.

3:30 Response of Structure to Explosive-Generated Ground Shock

GEORGE W. HOUSNER, A.M. ASCE, Professor of Engineering, California Institute of Technology.

Power Division

THEATER
FAIRMONT HOTEL

Presiding: H. V. Lutge, Member, Executive Committee, Power Division

2:00 Pacific Gas and Electric Co. Power Program

WALTER DREYER, M. ASCE, Vice-President and Chief Engineer, Pacific Gas and Electric Co.

2:45 Design Features—Pacific Gas and Electric Co. Hydroelectric Development

J. B. COOKE, JR., A.M. ASCE, Senior Civil Engineer, Pacific Gas and Electric Co.

Discussion: HERBERT W. HABERKORN, A.M. ASCE, Engineer of Hydroelectric Construction, Pacific Gas and Electric Co.

3:30 Cherry Valley Development of City of San Francisco

M. L. DICKINSON, M. ASCE, Chief Hydraulic Engineer, Power Division, Bechtel Corp.

Discussion: WESLEY F. GETTS, A.M. ASCE, Civil Engineer, Hetch Hetchy Water Supply, Engineering Bureau Power and Utilities, San Francisco, Calif.

Sanitary Engineering Division

NOB HILL ROOM
FAIRMONT HOTEL

Presiding: Harold B. Gotaas, Chairman, Executive Committee, Sanitary Engineering Division; and Richard R. Kennedy, M. ASCE, Program Committee, Sanitary Engineering Division

2:00 **Analysis of Water Quality Criteria**
JACK E. MCKEE, M. ASCE, Professor of Sanitary Engineering, California Institute of Technology.

2:45 **Control of Drinking Water Quality in Open Distribution Reservoirs**
BLAIR I. BURNSON, A.M. ASCE, Supervising Sanitary Engineer, East Bay Municipal Utility District.

3:30 **Alum Treatment of Storage Reservoirs to Reduce Turbidity During Floods**
RAY L. DERBY, M. ASCE, Principal Sanitary Engineer, Los Angeles Department of Water and Power; WM. K. WEIGHT, A.M. ASCE, Assoc. Sanitary Engineer, Los Angeles Department of Water and Power.

WEDNESDAY MORNING MARCH 4

Welcome to San Francisco and Business Session

TERRACE ROOM
FAIRMONT HOTEL

Presiding: Walter L. Huber, President, ASCE

10:00 **Welcome by San Francisco Section**
J. G. Wright, President, San Francisco Section, ASCE.

Welcome by City and County of San Francisco
Honorable Elmer E. Robinson, Mayor of San Francisco.

Welcome by State of California
Honorable Earl Warren, Governor of California.

Response
Walter L. Huber, President, ASCE.

11:00 **Century of Engineering in California**
Walter L. Huber, President, ASCE.

11:30 **General Business Session of ASCE**
Award of Daniel W. Mead Student Prize to Charles E. Negus, Jr., J.M. ASCE, Auburn, Calif.

MEMBERSHIP LUNCHEON

Wednesday, March 4 12:15 p.m.

GOLD ROOM
FAIRMONT HOTEL

Presiding: A M Rawn, Vice-President, ASCE

DR. ROBERT GORDON SPROUL, President, University of California, will give the principal address.

All members, their ladies and guests, and friends of ASCE are cordially invited to attend this luncheon.

Price, $3.50 per plate.

WEDNESDAY AFTERNOON MARCH 4

Air Transport Division

GARDEN ROOM
FAIRMONT HOTEL

Presiding: J. G. Bastow, M. ASCE, Chief Engineer, Port of Oakland

2:00 **Effect of Jet Aircraft on Airport Pavements**
GAYLE MCFADDEN, M. ASCE, Chief, Airfield Branch, Engineering Division, Office Chief of Engineers, Washington, D.C.; J. A. BISHOP, A.M. ASCE, Director, Soils and Pavement Div., U.S. Naval Civil Engineering Research and Evaluation Lab., Port Hueneme, Calif.

Discussion: C. E. RHODES, Director, Paving Section and Testing Div., Public Works Department, 12th Naval District.

3:00 **Determination of Radii of Curvature of Taxiways**
JOHN HUGH JONES, J.M. ASCE, Asst. Professor of Civil Engineering, University of California; and Asst. Engineer, Inst. of Transportation and Traffic Engineering; ROBERT HORONJEFF, A.M. ASCE, Lecturer and Research Engineer, Inst. of Transportation and Traffic Engineering, University of California.

Construction and Power Divisions, Joint Session

TERRACE ROOM
FAIRMONT HOTEL

Presiding: A. H. Ayers, Member, Executive Committee, Construction Division

2:00 **Civil Engineering Design Features in Pittsburgh and Contra Costa Steam Plants—Pacific Gas and Electric Co.**
WALTER L. DICKEY, A.M. ASCE, Chief Civil Engineer, Bechtel Corp.

2:45 **Some Construction Features in Pittsburgh and Contra Costa Steam Plants—Pacific Gas and Electric Co.**
EDGAR GARBARINI, A.M. ASCE, General Supt. of Construction, Bechtel Corp.

3:30 **Construction of Underwater Foundations for Transmission-Tower Line Crossing of San Francisco Bay Without Use of Cofferdams**
BEN C. GERWICK, JR., A.M. ASCE, Vice-President, Ben C. Gerwick, Inc., San Francisco, Calif.

Discussion: CARL W. APPLEFORD, M. ASCE, Supervising Civil Engineer, Pacific Gas and Electric Co.

Hydraulics Division

VANDERBILT ROOM
FAIRMONT HOTEL

Presiding: Thornton J. Corwin, Jr., Member, Executive Committee, Hydraulics Division

2:00 **Hydraulic Design of Pine Flat Dam**
AMOS W. HOGGARD, Supervising Hydraulic Engineer, Corps of Engineers, Sacramento District.

2:45 **Surge Chambers for Hydroelectric Plants**
J. D. WORTHINGTON, A.M. ASCE, Senior Engineer, Pacific Gas and Electric Co.

3:30 **Pressure Surge Control at Tracy Pumping Plant**
JOHN PARMAKIAN, Hydraulic Engineer, Bureau of Reclamation, Denver, Colo.

Sanitary Division

NOB HILL ROOM
FAIRMONT HOTEL

Presiding: Erman A. Pearson, A.M. ASCE, Chairman, Sanitary Engineering Committee, San Francisco Section

2:00 **Radioisotope Removal in Modern Waste Treatment**
WARREN J. KAUFMAN, J.M. ASCE, Asst. Professor of Sanitary Engineering, University of California; GERHARD KLEIN, Research Engineer, Institute of Engineering Research, University of California.

2:45 **New Sewage Treatment Facilities in San Francisco**
BEN BENAS, M. ASCE, Senior Engineer, Sewage and Waste Treatment Div., San Francisco Dept. of Public Works.

3:30 **East Bay Cities Sewage Disposal Project**
R. C. KENNEDY, M. ASCE, Chief Engineer, East Bay Municipal Utility District.

Structural Division

THEATER
FAIRMONT HOTEL

Presiding: Raymond Archibald, Chairman, Executive Committee, Structural Division

2:00 **Live Loading on Long-Span Highway Bridges**
NORMAN C. RAAB, M. ASCE, Projects Engineer, Division of San Francisco Bay Toll Crossings.

2:45 **Wind Load Tests on Highway Bridge Models**
GEORGE S. VINCENT, M. ASCE, Bridge Engineer, U.S. Bureau of Public Roads, Seattle, Wash.

3:30 **Stress Measurements on San Leandro Creek Bridge**
R. W. CLOUGH, J.M. ASCE, Asst.

Professor of Civil Engineering, University of California; C. F. SCHEFFEY, J.M., ASCE, Asst. Professor of Civil Engineering, University of California.

DINNER AND DANCE

Wednesday, March 4

VENETIAN, GOLD, NOB HILL ROOMS FAIRMONT HOTEL

7:00 p.m. Cocktails
Venetian Room

8:00 p.m. Dinner

9:30 p.m. Dancing and entertainment

Price, including cocktails, dinner and dancing, $8.50 per person. Dinner jacket optional.

The seating capacity is 950 persons. Reservations will be accepted in the order received.

THURSDAY
MARCH 5

CONVENTION EXCURSIONS

Buses leave from front entrance of the Fairmont Hotel at times scheduled below.

Tour No. 1
9:00 a.m.

Pittsburg Steam Plant, Pacific Gas and Electric Co., under construction.

Contra Costa Steam Plant, Pacific Gas and Electric Co., complete except for some machinery installation.

Luncheon: Complimentary.

Return via University of California, Materials Testing Laboratory.

Arrive Convention Headquarters, 5:00 p.m.

Price, $2.00.

Tour No. 2
9:30 a.m.

Limited to 250

Ames Aeronautical Laboratory, National Advisory Committee for Aeronautics, Moffett Field:
- Construction of 8-ft, supersonic wind tunnel
- Inspection of some of the existing wind tunnels

Luncheon: Complimentary.

Return via Stanford University.

Arrive Convention Headquarters, 5:30 p.m.

(Note: Members desiring to take this tour who are not United States citizens should make reservations well in advance, stating citizenship.)

Price, $2.00.

Tour No. 3
1:00 p.m.

Limited to 150

San Francisco Naval Shipyard:
- Dry docks, huge fitting-out crane, and completely equipped shops
- Reserve (mothball) fleet of cruisers and carriers
- Submarine visit

Arrive Convention Headquarters, 4:30 p.m.

(Note: Members desiring to take this tour who are not United States citizens should make reservations well in advance, stating citizenship.)

Price, $1.00.

THURSDAY EVENING
MARCH 5

No organized entertainment is provided for Thursday night, as it is thought that visitors would prefer to have an open evening to visit interesting places of their own selection in San Francisco.

FRIDAY MORNING
MARCH 6

STUDENT CHAPTER FACULTY ADVISERS' CONFERENCE

FAIRMONT HOTEL **9:30 a.m.**

Faculty Advisers from ASCE Student Chapters in the West will convene for a one-day discussion of Student Chapter problems.

This Conference, which is primarily for invited Advisers, will be open to all Contact and Junior Contact Members of Chapters and others interested in the activities and operational details of ASCE Student Chapters.

Highway Division

THEATER
FAIRMONT HOTEL

Presiding: Ralph A. Moyer, A.M. ASCE, Chairman, Highways Committee, San Francisco Section

9:00 **Design and Construction of Broadway Tunnel**
RALPH G. WADSWORTH, M. ASCE, City Engineer, San Francisco, Calif.

9:45 **Economic Studies in Location and Design of Richmond—San Rafael Toll Bridge—a $62 Million Project**
NORMAN C. RAAB, M. ASCE, Projects Engineer, Division of San Francisco Bay Toll Crossings; RALPH A. TUDOR, W. ASCE, Consulting Engineer, San Francisco, Calif.

10:30 **Economics of Bank Protection Structures to Prevent Erosion of Highways**
R. ROBINSON ROWE, M. ASCE, Supervising Bridge Engineer, California Division of Highways.

11:15 **California Experience in Correction of Highway Landslides and Slipouts**
A. W. ROOT, A.M. ASCE, Supervising Materials and Research Engineer, California Division of Highways.

Hydraulics Division

VANDERBILT ROOM
FAIRMONT HOTEL

Presiding: J. K. Vennard, A.M. ASCE, Chairman, Hydraulics Committee, San Francisco Section

9:00 **Symposium: West Coast Hydraulic Research Activity at:**
California Institute of Technology
Oregon State College
Stanford University
State College of Washington
University of California at Berkeley
University of Washington

Irrigation and Drainage Division

GARDEN ROOM
FAIRMONT HOTEL

Presiding: Harry F. Blaney, Member, Executive Committee, Irrigation and Drainage Division

9:00 **Solutions for Irrigation Problems in Alameda Creek Basin**
HERBERT CROWLE, A.M. ASCE, Chief Engineer, Alameda County Flood Control and Water Conservation District.

Discussion: JOHN M. HALEY, A.M. ASCE, Supervising Hydraulic Engineer, Division of Water Resources, California Department of Public Works.

10:00 **Some Irrigation and Drainage Problems of Mediterranean and Near East Countries**
MARTIN R. HUBERTY, A.M. ASCE, Professor of Irrigation, University of California, Los Angeles, Calif.

11:00 **Use of Colorado River Water in California**
RAYMOND MATTHEW, M. ASCE, Chief Engineer, Colorado River Board of California.

Discussion: M. J. DOWD, M. ASCE, Consulting Engineer, Imperial Irrigation District.

Soil Mechanics and Foundations Division

NOB HILL ROOM
FAIRMONT HOTEL

Presiding: R. F. Blanks, Member, Executive Committee, Soil Mechanics and Foundations Division

9:00 Soil Mechanics and Geology—with Special Reference to San Francisco Area

PARKER D. TRASK, Consulting Geologist, Berkeley, Calif.

Discussion: HARRY A. WILLIAMS, A.M. ASCE, Assoc. Professor, Civil Engineering Department, Stanford University.

10:30 Building Foundations in San Francisco

CHARLES H. LEE, M. ASCE, Consulting Engineer, San Francisco, Calif.

Discussion: HENRY J. BRUNNIER, M. ASCE, Consulting Structural Engineer, San Francisco, Calif.; and WM. W. MOORE, M. ASCE, Dames and Moore, San Francisco, Calif.

Structural Division

TERRACE ROOM
FAIRMONT HOTEL

Presiding: George E. Brandow, Member, Executive Committee, Structural Division

9:00 Structural Observations of the Kern County Earthquakes in 1952

HENRY J. DEGENKOLB, M. ASCE, Chief Engineer; and JOHN J. GOULD, Consulting Engineer, San Francisco, Calif.

10:00 Heaviest Prestressed Building Girders in United States

T. Y. LIN, M. ASCE, Assoc. Professor of Civil Engineering, University of California.

11:00 Structural Welding Inspection

J. L. BEATON, A.M. ASCE, Supervising Highway Engineer; and PAUL JONAS, Welding Engineer, Materials and Research Department, California Division of Highways.

MILITARY ENGINEERS' LUNCHEON

Friday, March 6 **12:15 p.m.**

VENETIAN ROOM
FAIRMONT HOTEL

Sponsored by the San Francisco Post, Society of American Military Engineers. All members of ASCE and their guests are invited. Speaker—an outstanding military figure (to be announced later).

Tickets, $3.50 per plate.

FRIDAY AFTERNOON
MARCH 6

Highway Division

THEATER
FAIRMONT HOTEL

Presiding: Roy E. Jorgensen, Chairman, Program Committee, Member Executive Committee, Highway Division

2:00 Appraisal of One-Way Street Systems

D. J. FAUSTMAN, A.M. ASCE, Traffic Engineer, Sacramento, Calif.; and F. T. FOWLER, M. ASCE, Traffic Engineer, Portland, Oreg.

2:45 How California Cities Are Solving Their Parking Problems

W. H. KENNEDY, Assoc. Engineer, Inst. of Transportation and Traffic Engineering, University of California.

3:30 Highway Engineering Education Program at University Level

H. E. DAVIS, A.M. ASCE, Director; and R. A. MOYER, A.M. ASCE, Professor of Civil Engineering and Research Engineer, Inst. of Transportation and Traffic Engineering, University of California.

4:15 Redesign of Intersections on Arterial Highways

E. T. TELFORD, M. ASCE, Traffic Engineer, California Division of Highways.

Irrigation and Drainage Division

GARDEN ROOM
FAIRMONT HOTEL

Presiding: Harry F. Blaney, Member, Executive Committee, Irrigation and Drainage Division

2:00 Operation of Central Valley Project

MARTIN H. BLOTE, M. ASCE, Superintendent, Central Valley Project Operations, Bureau of Reclamation, Sacramento, Calif.

Discussion: T. B. WADDELL, M. ASCE, Asst. State Engineer, Division of Water Resources, California Department of Public Works.

3:00 Statutory Control of Ground Water Draft

S. T. HARDING, M. ASCE, Consulting Engineer, Berkeley, Calif.

Discussion: HAROLD CONKLING, Member, Executive Committee, Irrigation and Drainage Division.

4:00 California State-Wide Water Resources Investigation

A. D. EDMONSTON, M. ASCE, State Engineer, Div. of Water Resources, California Department of Public Works, Sacramento, Calif.

Discussion: A. N. MURRAY, A.M. ASCE, Regional Planning Engineer, Bureau of Reclamation, Sacramento, Calif.

Soil Mechanics and Foundations Division

NOB HILL ROOM
FAIRMONT HOTEL

Presiding: R. F. Blanks, Member, Executive Committee, Soil Mechanics and Foundations Division

2:00 Design and Construction Problems for Rock-Filled Dams

DONALD J. BLEIFUS, A.M. ASCE, International Engineering Co., Inc.; and J. P. HAWKE, A.M. ASCE, International Engineering Co., Inc.

Discussion: I. C. STEELE, M. ASCE, Consulting Engineer, San Francisco, Calif.

3:00 Geology and Dams in California

ELMER MARLIAVE, Geologist, Division of Water Resources, California Department of Public Works, Sacramento, Calif.

Discussion: HYDE FORBES, M. ASCE, Engineer and Geologist, Palo Alto, Calif.

4:00 Foundation Treatment for Earth Dams on Rock

THOMAS F. THOMPSON, Aff. ASCE, Chief of Geologic Section, South Pacific Division, Corps of Engineers.

Discussion: WM. GARDNER, Geologist, U.S. Bureau of Reclamation, Sacramento, Calif.

Surveying and Mapping Division

VANDERBILT ROOM
FAIRMONT HOTEL

Presiding: W. H. Rayner, Member, Executive Committee, Surveying and Mapping Division

2:00 Cooperative Topographic Mapping Program in California

C. A. ECKLUND, M. ASCE, Pacific Region Engineer, Topographic Division, USGS, Sacramento, Calif.

Discussion: TRACY L. ATHERTON, Topographic Engineer, California Division of Water Resources, Sacramento, Calif.

3:00 Triangulation and Lambert Coordinate System for California

LANSING G. SIMMONS, M. ASCE, Chief Mathematician, U.S. Coast and Geodetic Survey, Washington, D.C.

Discussion: F. H. MOFFITT, Asst. Professor of Civil Engineering, University of California.

Waterways Division

TERRACE ROOM
FAIRMONT HOTEL

Presiding: W. O. Hiltabidle, Chairman, Executive Committee, Waterways Division

2:00 **Fish Rescue Project at Pacific Gas and Electric Co. Contra Costa Steam Plant**

JAMES E. KERR, A.M. ASCE, Senior Engineer, Bechtel Corp.

2:45 **Model Studies for Apra Harbor, Guam, and Los Angeles Harbor**

ROBERT T. KNAPP, M. ASCE, California Institute of Technology; and VITO A. VANONI, M. ASCE, California Institute of Technology.

Discussion: J. W. JOHNSON, M. ASCE, Prof. of Civil Engineering, University of California.

3:30 **Salt Water Barriers in San Francisco Bay**

BYRON L. NISHKIAN, M. ASCE, Consulting Engineer, San Francisco, Calif.; and PAUL CUSHING, A.M. ASCE, President, Hydraulic Dredging Co., Oakland, Calif.

Discussion: DONALD J. BLEIFUS, A.M. ASCE, International Engineering Co., Inc.

POST CONVENTION TOUR TO HAWAII

Tuesday, March 10

Registration: 9:30 to 10:30 a.m., at University of Hawaii Administration Building. Fee, $3.00, except ladies and students.

11:00 **Opening Session**

ANDREWS AMPHITHEATER
UNIVERSITY OF HAWAII

Presiding: George C. Wallace, President, Hawaii Section

Address of welcome

Governor of the Territory.

Noon Luncheon

Host, University of Hawaii Student Chapter. Price, $1.50 per plate.

2:00 **Technical Meeting, University of Hawaii**

Bridges on the Hamakua Coast Highway

ROBERT W. BELT, M. ASCE, Superintendent of Public Works and Territorial Highway Engineer.

Construction of Honolulu's Outfall Sewer

GEORGE C. WALLACE, M. ASCE, Engineer, Division of Sewers, Department of Public Works, City and County of Honolulu.

Basal Water Development in Hawaii

LESLIE J. WATSON, M. ASCE, Engineer, Division of Water Resources, Honolulu Board of Water Supply.

Wednesday, March 11

9:00 **Field inspection trip**

Nuuanu Pali and South Shore Drive

Halawa underground water pumping plant

12:30 **Luncheon**

Pearl Harbor Naval Reservation. Price, $1.50 per plate.

2:00 **Tour of Pearl Harbor Naval Reservation**

6:00 **Hawaiian Luau (feast)**

Includes cocktails and Hawaiian entertainment. Informal or Hawaiian dress. Price $10.00 per plate for everything.

Reservations: As this will be the height of the tourist season, reservations for travel and hotel should be made early. Handling arrangements for the Hawaiian Section is:

Mr. George R. Smith
210 Post St.
San Francisco 8, Calif.

A deposit of $50.00 should accompany requests for Hawaiian travel and hotel reservations.

LADIES' ENTERTAINMENT

Ladies Headquarters:

CALIFORNIA ROOM
MEZZANINE FLOOR
FAIRMONT HOTEL

The California Room will be the starting point of all events. It will be open 9:00 a.m. to 5:00 p.m. Monday through Friday. Hostesses will be in attendance to serve coffee in the morning and tea in the afternoon and to answer questions.

Tuesday, March 3

9:30 a.m. Peninsula sight-seeing trip
Sunset Magazine headquarters
Luncheon at Allied Arts Guild
Stanford University
Price, $3.50 per person

Wednesday, March 4

9:30 a.m. Coffee hour, California Room

12:15 p.m. Membership luncheon, Gold Room, Fairmont Hotel

7:00 p.m. Cocktails and dinner dance, Venetian, Gold and Nob Hill Rooms, Fairmont Hotel

Thursday, March 5

9:45 a.m. San Francisco sight-seeing trip
Tour through part of city

Main stop at McClellan's Nursery, famous for air-expressing orchids, gardenias and heather

Luncheon at Olympic Lakeside Country Club

Price, $4.00 per person

Friday, March 6

12:30 p.m. Luncheon and Aquacade
Tonga Room, Fairmont Hotel

Featuring the Crystal Plunge Swimming Team including Olympic Games participants

Price $4.00 per person

STUDENT CHAPTER CONFERENCE

Monday, March 2

9:00 a.m. Registration (free to students and their wives) and housing.
Lobby, Fairmont Hotel.

Noon "Meet-the-Board" luncheon
Venetian Room, Fairmont Hotel

2:00 p.m. Student paper competition
Nob Hill Room, Fairmont Hotel

7:00 p.m. "Get acquainted" dinner and smoker
Stag. Place to be announced.
Half of dinner costs to be borne by Convention.

Price $2.00, per student.

Tuesday, March 3

9:00 a.m. Student conference and forum on Chapter affairs
Nob Hill Room, Fairmont Hotel

Throughout the regular San Francisco Convention, students are cordially invited and encouraged to attend Technical Division sessions and other Convention events.

AUTHORS' LUNCHEON AND BREAKFASTS

Briefing sessions for all speakers, discussers and program officials by invitation.

Presiding: Carl A. Trexel, Chairman, Technical Program Committee

Luncheon
TUESDAY, MARCH 3, 12:00 P.M.
Garden Room, Fairmont Hotel

Breakfast
WEDNESDAY, MARCH 4, 8:00 A.M.
Garden Room, Fairmont Hotel

Breakfast
FRIDAY, MARCH 6, 8:00 A.M.
Cirque Room Lounge, Fairmont Hotel

CALIFORNIA SECTIONS CONFERENCE

Wednesday, March 4, 4:30 p.m.

GARDEN ROOM
FAIRMONT HOTEL

Presiding: John S. Longwell, Past-President, San Francisco Section

CONVENTION COMMITTEES

L. A. ELSENER, *General Chairman*

General Convention Committee

James I. Ballard — Harry H. Hilp
Walter Dreyer — I. C. Steele
L. A. Elsener — Glenn Woodruff
J. G. Wright

Hotels and Registration
W. W. MOORE, *Chairman*

Technical Program
C. A. TREXEL, *Chairman*

Entertainment
W. H. ARATA, *Chairman*

Student Activities
B. A. VALLERGA, *Chairman*

Excursions
H. W. HABERKORN, *Chairman*

Reception
S. P. DUCKEL, *Chairman*

Publicity
L. L. WISE, *Chairman*

Local Sections' Conference
J. E. RINNE, *Chairman*

Finance
J. P. YATES, *Chairman*

Military Engineers' Luncheon
L. S. MOELLER, *Chairman*

Hawaiian Post-Convention Tour
R. M. BELT, *Chairman*

Ladies' Activities
MRS. L. A. ELSENER, *Chairman*

HOTEL RESERVATIONS

March 3 through March 6, 1953

A Housing Bureau has been organized for the Convention in San Francisco. Since all requests for rooms are handled in chronological order, it is recommended that you send in your application as quickly as possible. The Fairmont Hotel will be the Convention Headquarters and a certain number of rooms will be held there and in two adjacent hotels, the Mark Hopkins and Huntington Hotels.

In making hotel reservations, please use the blank on page 90. Indicate your first, second, and third choice hotel. Because of the limited number of single rooms available, you will have a better chance of securing accommodations in a hotel of your choice, if your request calls for rooms to be occupied by two or more persons.

All requests should be accompanied by a deposit check of $5.00 per person (or a minimum of $10.00 per room), made out to the ASCE Housing Bureau. Because of existing crowded conditions, hotels cancel unclaimed reservations by 6:00 p.m. Therefore, a deposit is requested to insure your reservation will be held on your arrival day—whatever the hour—and will be credited to your account. **Please do not send cash.**

Please complete the application on page 90, giving all information requested. Note that this is for hotel accommodations; convention registration is separate—the form is on page 88.

All reservations will be confirmed if request is received not later than February 1.

Information and Registration

Members are urged to pre-register to assure reservations at important events that are necessarily limited as to attendance. Use form on p. 88 of this issue for pre-registration.

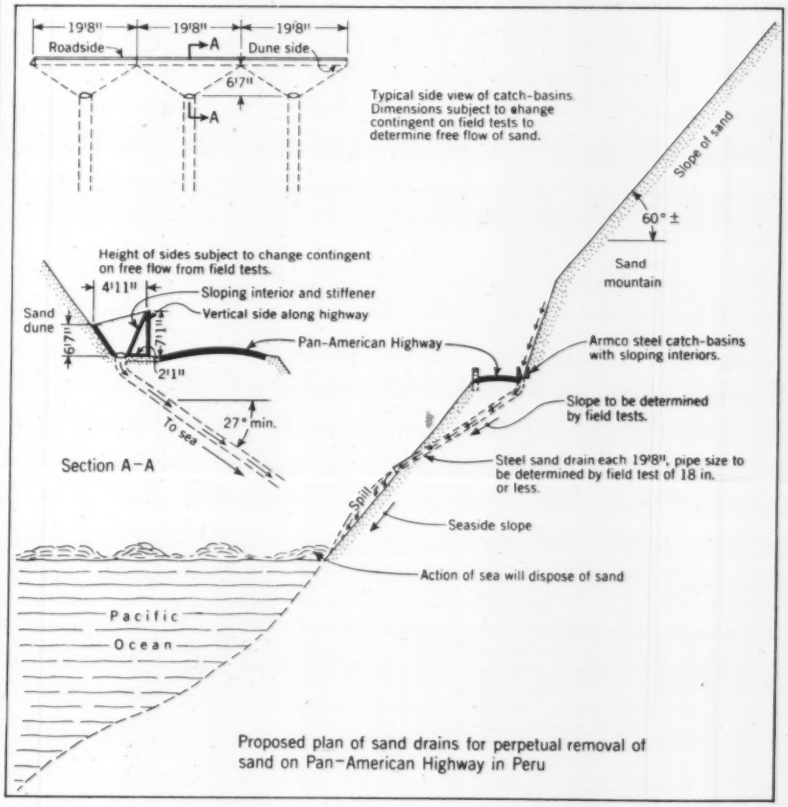
The registration desk will be open in the lobby, Fairmont Hotel, during the Convention for information service. Mail and messages will be held there for members.

Press Room

EMPIRE ROOM ANNEX

For the convenience of the technical press, newspapers and radio, a press room will be open throughout the Convention. Members of the publicity committee will be on hand during Convention sessions to provide information to the press.

Solution to problem on page 63



Proposed plan of sand drains for perpetual removal of sand on Pan-American Highway in Peru

Notes:

1. Practicality of driving pipe in short sections using special ends to be tested in field. If this is possible, traffic can continue as usual during construction.
2. Specially built wood or steel catch-basins ean be used in lieu of Armco units.
3. Pipe sizes ranging from 6 to 18 in. to be tried.
4. Various types of pipe to be tested, including Transite.
5. Glass liners to be considered for pipe.
6. Plan to be used only in critical sections where constant spills are encountered.

SOCIETY NEWS

San Francisco Convention

March 2–7, 1953

Here San Francisco Bay Bridge completes its giant strides from the East Bay to San Francisco's Embarcadero. City skyline is visible in background.

These San Francisco vistas suggest a reason for attending the Society's San Francisco Convention, March 2–7, almost as compelling as the program of Technical Division sessions, social events, and inspection trips printed elsewhere in this issue. Said to be one of the three most colorful cities in the United States, San Francisco is an interesting and exhilarating combination of the quaint (as exemplified by its cable cars, Chinatown, and Fishermen's Wharf) and the modern. It is a city of endless views and of a sense of the nearness of the sea, despite its cosmopolitan aspect, its world-famous hotels and restaurants, and its fascinating stores and shops. To assure Convention visitors a chance to do some sightseeing on their own, Thursday evening has been left open. The ladies' program will include tours of both the city and the peninsula.

For engineers, in addition to the previously announced all-day inspection trips to the new Pittsburg Steam Plant and the 8-ft supersonic wind tunnel under construction at the Ames Aeronautical Laboratory, a half-day excursion to the San Francisco Naval Shipyard has been arranged. The tour will include inspection of drydocks, the completely equipped shops with their huge fitting-out crane, and the reserve fleet of carriers and cruisers and a submarine visit. Possibly even more interesting will be the tour of the rebuilt facilities at the Pearl Harbor Naval Yard on the agenda for those making the post-convention trip to Hawaii.

Although the Golden Gate and San Francisco–Oakland Bay bridges are not new, they are perennially interesting from an engineering standpoint as well as perennially beautiful. In Union Square, San Francisco, engineers will have a chance to observe one of the first major municipal parking ventures carried out underground in the United States.

The rich technical program, which is being readied under the chairmanship of Rear-Admiral C. A. Trexel, will include a symposium of six papers reporting West Coast hydraulic research activity at six West Coast technical institutes.

California Street, vital nerve in San Francisco's business district, is seen in left-hand photo. Dropping precipitously from Nob Hill, between Fairmont and Mark Hopkins hotels, it passes beside Chinatown and crosses the financial center. View at right shows the Fairmont Hotel on Nob Hill, ASCE Convention headquarters. Famous Mark Hopkins Hotel is directly across the street.

Manual on Concrete Shell Roofs Available

ASCE Manual 31, entitled "Design of Cylindrical Concrete Shell Roofs," presents to the engineering profession, for the first time, an extensive set of tables and charts that enable an engineer to design shell roofs without spending innumerable days on tedious numerical computations. By means of coefficients tabulated in the tables, the final moments and forces acting throughout the shell are obtained in two or three slide-rule operations. In consequence, shells can be designed in a matter of hours for even complicated layouts.

This ease in design accomplishes two things. First, it brings the design of shell roofs within the realm of the average practicing engineer. Second, it permits a more thorough evaluation of the economics of column arrangement for shell roofs. Another important advantage resulting from this systematic array of values is that the effect of the several variables, such as chord width, span length, thickness, and curvature of shell can be quickly studied. Because of this, the structural behavior of shells will be better understood.

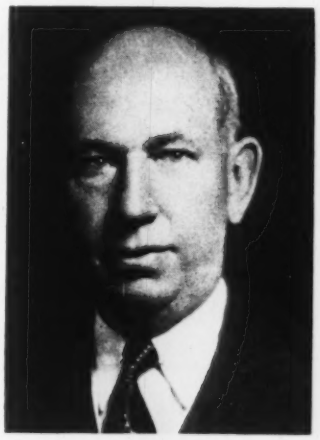
A 177-page, paper-bound volume, Manual 31 is available at Society Headquarters at $5 a copy, with a 50 percent discount available to members of ASCE. A convenient form for ordering the manual is given on page 109 of the advertising section of this issue.

Sydney Wilmot Retires From Publication Staff

On December 31, Sydney Wilmot is retiring as Manager of ASCE Technical Publications after almost thirty years on the Society's staff. Importantly identified with the production of Society publications since 1923 when he joined the headquarters staff, Mr. Wilmot has been successively Technical Editor, Manager of Publications, and Manager of Technical Publications. In these capacities, he has directed the publishing of PROCEEDINGS, TRANSACTIONS, CIVIL ENGINEERING, PROCEEDINGS–SEPARATES, Technical Manuals, Manuals of Engineering Practice, and other occasional Society publications—the entire written record of ASCE technical accomplishment for the period.

In the late twenties, Mr. Wilmot was one of a small group of staff and Board members who conceived and worked out plans for a civil engineering magazine of broader general and less strictly technical appeal. When the plans for the new publication came to fruition in 1930 with issuance of CIVIL ENGINEERING, Mr. Wilmot supervised its editing and publishing—a post he held for sixteen years as Manager of Publications. A small staff of key personnel which he built up is still in responsible charge of producing the magazine. Along with his other duties, he has also continuously served as secretary of the Society's Committee on Publications for twenty-four years.

A graduate of Brown University with the degree of B.S. in C.E., Mr. Wilmot received the A.M. degree from Columbia University in 1913. His early experience covered work with the Catskill Water Supply project for New York, the New York subway system, and the building of the Panama Canal—the largest individual engineering undertakings in the United States up to that time. From 1918 to 1923 he was assistant professor of civil engineering at Brown University. Prominent in the Brown Engineering Association, he served it as president in 1927 and, again, in 1946.



Sydney Wilmot

Mr. Wilmot's membership in ASCE goes back to 1909, when he became a Junior. He transferred to Associate Member in 1919, and has been a Member since 1923. He is also a member of the Society of Sigma Xi and of Chi Epsilon fraternity, honorary technical organization. He has served as a member of the national missionary organization of the Baptist Church, has been prominent in local church affairs, and since 1940 has served as president or trustee of the Jennie Clarkson Home for Children, Valhalla, N.Y. In his retirement, he will have the good wishes of the host of friends he has made in his work. He will continue to live at Tuckahoe, N.Y., where he will undertake consulting work on special problems concerned with the editing and publication of technical papers and reports.

Library Serves Many By Mail and Telephone

The Engineering Societies Library served more engineers by mail and telephone in the 1951–1952 fiscal year than in any previous year of its existence, according to its recently released 39th annual report. Inquiries from 21,255 non-visitors were received and answered. This represented a considerable increase over the 16,783 non-visitors served in the 1950–1951 fiscal year and the first time more non-visitors than visitors (numbering 17,585) were served.

Other services performed by the Library during the year include the filling of 4,381 photostat orders; the making of 52,003 photostat prints; 176 microfilm orders; 206 bibliography orders; 153 searches and paid services; and 184 translations with a total of 310,838 words. There were 1,943 borrowers of 2,803 books; 11,092 telephone inquiries; and 3,120 letters written (exclusive of book orders).

Miscellaneous activities included the review of 484 books valued at $2,770. These reviews are supplied to the four Founder Societies, the Engineering Institute of Canada, and the Engineering Index. In addition to the books received for review, $1,947 was spent on books that were not reviewed. A new Engineering Societies Monograph, entitled "The Buckling Strength of Metal Structures" by the late Friedrich Bleich, M. ASCE, was published (February issue, page 57). The monograph series is controlled by a separate committee of the Founder Societies rather than by the Library Board. The snow, ice, and permafrost bibliography (June issue, page 65) being received from the Library of Congress in return for help given it has grown from 600 to 3,000 references during the year.

Under joint sponsorship of the four Founder Societies, the Library is prepared to conduct searches, make translations and perform multiple other services at nominal rates. Dr. Ralph H. Phelps is head of the Library.

Texas Section Performs Generous Public Service

A notable public service has been rendered the citizens of Texas by the Texas Section in conducting a year-long study on the water situation in the state and presenting the results in a comprehensive report, entitled, "A Water Policy for Texans." In a letter transmitting the report to Gov. Allan Shivers, T. C. Forrest, Jr., the president, characterized the report as "a sound document which the State of Texas can use as a guide and thus retain control over its water resources and the useful application thereof."

Recommendations made by the Texas Section include a new surface water code for Texas; local option districts for regulation of underground waters; a state agency for enforcement of pollution laws; appropriation of adequate authority and funds to the State Board of Water Engineers; a state policy with respect to federal activities based on state ownership of its water resources; amendment to the National Reclamation Act, designed for public land states, so that the U.S. Bureau of Reclamation can contribute more effectively to solution of Texas water problems; and provision for material cooperation with government agencies in flood control, soil conservation, and stream gauging activities.

The Section's report condemns, and recommends amendment of, federal laws which require a 50-year recapture clause under license from the Federal Power Commission for projects on navigable streams. The report vigorously recommends state support for the principles contained in the Engineers Joint Council report on Federal Water Policy, and comes out strongly in favor of compacts with adjacent states for the amicable allocation of the water of interstate streams. As to financial assistance by the State, the Texas Section maintains that "full responsibility rests with the political subdivision being assisted, including eventual repayment by it of state funds."

Governor Shivers expressed the gratitude of the state on the occasion of the formal presentation of the report and recommended it for full consideration of a statewide committee he has appointed to study the critical water shortage in Texas, which has developed as a result of the prolonged drought. The Section's Committee on State Water Policy, which prepared the report, included Simon W. Freese, chairman, James A. Cotton, Mason G. Lockwood, Harry N. Roberts, and Robert A. Thompson, Jr.

A 17-page summary of the report is available at 50 cents a copy. Inquiries should be addressed to the secretary of the Section, I. W. Santry, Jr., Southern Methodist University, Dallas 5, Tex.

Formal presentation of Texas Section's water report is made to Governor of Texas in executive offices in Austin. Shown, left to right, are Marvin C. Nichols, member of Governor's statewide committee; L. D. Snow, past-president of Texas Section and chairman of its Public Relations Committee; T. C. Forrest, Jr., immediate past-president of Section and member of Governor's committee; Governor Allan Shivers; Mason P. Lockwood, president of Section and member of both Texas Section and Governor's committees; and Robert A. Thompson, member of Texas Section committee.

Engineers Joint Council Actions Briefed

Actions of the Engineers Joint Council which met in the Engineering Societies Building in New York on November 21, with President T. G. Le Clair in the chair, are abstracted here.

Permanent Secretary

On recommendation of the Executive Committee, T. A. Marshall, Jr., executive secretary of the Engineering Manpower Commission, was appointed secretary of Engineers Joint Council. Mr. Marshall will be the first secretary of the organization on a permanent basis.

T. A. Marshall, Jr.

Certificate of Incorporation

A draft of a Certificate of Incorporation for EJC, which had been prepared by legal counsel, was approved for transmittal to the constituent societies.

Engineering Manpower Commission

Steps will be taken to correct some of the inequities in the Armed Forces Reserve Act by appropriate Congressional legislation, and to work for better utilization of engineers in industry and in the Armed Forces.

It was reported that 1952 freshman enrollment in engineering is 29 percent higher than in 1951, while all freshman enrollment increased only 11 percent. Engineers in the graduating class of 1956 should number about 29,000 if Selective Service allows them to finish school.

Increased Unity in Engineering Profession

Constituent societies of EJC have approved amending the EJC constitution to permit its enlargement by inviting seven other societies to become members. The seven are: The American Association of Engineers; American Society for Engineering Education; American Society of Heating and Ventilating Engineers; the Institute of Aeronautical Sciences; the Institute of Radio Engineers; National Society of Professional Engineers; and Society of Naval Architects and Marine Engineers.

FROM THE NATION'S CAPITAL

JOSEPH H. EHLERS, M. ASCE — Field Representative ASCE

More conservative spending, radical relaxation of price and wage controls, greater administrative direction by the states—all seem to be indicated by the cabinet designations of the President-elect. The appointment of Gov. Douglas McKay of Oregon as Secretary of the Interior gives promise of handling reclamation development along lines advocated by engineers. The great needs of the West for water and power will be supported, but with considerable administrative direction probably left in the hands of the states.

It is expected that spending will be curtailed but not enough to depress the economy or to afford any magical tax relief. NPA controls will likely have rough sledding.

NPA Controls

Curbs on construction have been eased by NPA, largely by a shift from May 1 to January 1 for some previously scheduled relaxations. The two-year-old ban on recreational construction was lifted, permitting some self-authorization of structural shapes per project. Highways may use 12 tons of shapes per project. On other types of construction increased amounts of steel may be self-authorized. For example, industrial, commercial and public construction, water and sewage projects may use 25 tons of steel through the self-authorization procedure. Barring unforeseen developments, controls seem to be on the way out. Steel fabricators are still worrying about where to find the customers.

Outlook for Architects

Although engineers have been fully employed for some time, many architectural offices have been worrying about 1953 prospects. A recent survey, however, shows architects' offices also at an all-time peak of activity with continuance of favorable conditions predicted.

Washington Office Activities

Operations of the Washington office are largely confined to national affairs in the broader sense, including the whole range of government activity from highly technical engineering developments to broad aspects of government affecting the engineer's welfare. Some idea persists that the Washington office is a regional office with special interest in the Eastern States, because formerly there were three such offices—in Los Angeles, Chicago, and Washington—with certain regional aspects in the services rendered. However, since the closing of the other two offices and especially since the start of the Korean fighting, activities of nationwide interest have expanded so greatly that this office has dealt largely with strictly national affairs, with consequent dropping of services of a regional nature. Its work should be of as much interest to members in the West as in the East, and suggestions for making it so, in respect to the content of this column or otherwise, will always be welcomed.

In some future issue, detailed activities will be related, but for the present it is enough to say that our interests cover the entire range of ASCE membership. We actively participate in matters of fees, contract provisions, renegotiation, and materials controls, because we desire to aid members engaged in consulting practice or in contracting.

We concern ourselves with welfare problems such as labor laws and exemption from salary stabilization restrictions in order to be helpful to our employee members. We become involved in Civil Service procedures because we want the federal service to provide satisfying careers for engineers. As a matter of fact, the Advisory Committee on Engineers is the only Advisory Committee now functioning in the U.S. Civil Service Commission. Research work of government agencies is often followed because research is a vital tool to many members. The overseas engineering operations of government agencies are of interest to us because we believe there is a place for American civil engineers throughout the world. These office activities cover interests of all age groups from the young graduate and junior engineer right through to those of the largest and most experienced firms.

Section Centennial Celebrations

The Centennial of Engineering has furnished the theme for important local meetings and has provided a stimulus for gatherings of great interest in the District of Columbia and nearby areas which the Field Representative has attended recently. The Commissioners of the District of Columbia made an official proclamation of Centennial of Engineering Week on the occasion of the local Centennial Celebration. The seven engineering wonders of the National Capital were named by the local engineers. At the December meeting of the Virginia Section an important address outlining the development of engineering in the state of Virginia during the past century was given. The Maryland Section had remarkable turnouts for its enthusiastic meetings, as for example an attendance of well over 50 percent of the 300 members residing in Baltimore for its December meeting.

Washington, D.C.

December 15, 1952

District Nine Award Honors Dean Terrell

An annual competition for Junior Members in District 9 has been established by the District 9 Council in honor of former ASCE Vice-President Daniel V. Terrell. The award, first of its type to be established by one of the District Councils of Local Sections, recognizes Dean Terrell's "many years of work and service to the Society and, especially, his leadership in the establishment of the District 9 Council."

The award, consisting of a bronze plaque suitably mounted on a wooden shield, is to be given to a Junior Member in the District for a paper on a subject to be designated by the Competition Committee. The subject selected for the 1953 competition is "ASCE—My Observations of the Society." Contestants must conform to the following rules:

1. No individual may submit more than one paper in any one annual competition.

2. The paper must be of such length that oral presentation will not require more than 15 minutes.

3. To be eligible for the final competition, to be held during the annual convention of the District 9 Council, the paper must have been presented before the contestant's Local Section. Each Section will be restricted to one nominee in the final competition—the Section to submit the paper selected to the chairman of the Competition Committee prior to the Council's annual convention. All papers must be submitted in triplicate.

Latest developments in prestressed concrete construction are discussed by Jean Muller and James Libby, of Freyssinet Co., at December 3 dinner meeting of Oregon Section in Portland. Shown, left to right, are former ASCE Vice-President John W. Cunningham; H. Loren Thompson, vice-president of Oregon Section; Gordon L. Burt, secretary of Oregon Section; Charles Johnson, first Oregon contractor to use prestressed concrete construction; Mr. Muller; Thomas Smithson, Oregon Section chairman; Mr. Libby; and Hal W. Hunt (standing), who is currently supervising development of Port of Portland.

Details of Freeman Fellowship Given

The far-reaching aims of a famous American engineer, the late John R. Freeman, Past-President of both ASCE and ASME, are being implemented once again by the offering of another Freeman Fellowship. The purpose of the present award remains the same as when the fellowships were established in 1924—to aid and encourage young engineers in promoting the science and practice of hydraulic engineering.

Alternately conducted by ASCE and ASME, the 1953 award for a study or research project to be completed in 1954 is being arranged by an ASCE committee under the charimanship of M. P. O'Brien, member of both societies. The committee has established the following rules:

1. Applicants must submit a program of study or research in hydraulics or related fields, covering a period of at least nine months starting in 1953. A statement of funds needed from the fellowship should accompany applications.

2. Applicants must furnish evidence of qualification to carry out the proposed program.

3. Applicants must be citizens of the United States and members in some grade of either of the two cooperating societies.

4. The Freeman Award Committee will give preference to projects bearing importantly on the defense effort.

5. Applications should be submitted to the Freeman Fund Committee, care of the Executive Secretary, ASCE, 33 West 39th Street, New York 18, N.Y., by February 1, 1953. Announcement of the award will be made on March 15, and the recipient must make a report in English within 60 days of completion of his project.

The amount of grant from the Freeman Fund to a winning applicant has been $2,500.

Texas Section Forms Two More Branches

To better serve its far-flung membership, the Texas Section has established two more Branches—the West Texas Branch with headquarters at Midland and the High Plains Branch incorporating the Lubbock, Amarillo, and Plainview areas.

Officers for the West Texas Branch will be Henry E. Meadows, president; Henry E. Nunn, vice-president; and Banks McLaurin, Jr., secretary-treasurer. All are in Midland. The High Plains Branch officers are H. N. Roberts, Lubbock, president; G. C. Hatfield, Amarillo, vice-president; and George Whetstone, Lubbock, secretary-treasurer. Winfield Holbrook, of Plainview, will be Branch director to the Section.

Establishment of these groups brings to twelve the total of Texas Section Branches. Newly elected officers for the other ten Branches are listed in the December issue.

Coming Events

Central Ohio—Dinner meeting in the Ohio Union Building, Ohio State University campus, January 22, at 6:30 p.m.

Iowa—Meeting in connection with the annual meeting of the Iowa Engineering Society at Des Moines, February 10, at 10 a.m.

Kansas City—Dinner meeting at the Wishbone Restaurant, January 13, at 6:30 p.m., preceded by a social hour at 5:30 p.m. Business meeting at 7:30 p.m.

Los Angeles—Dinner meeting at the Alexandria Hotel, Los Angeles, January 14, at 6:30 p.m. Junior Forum meeting at the Alexandria Hotel, January 14, at 5:45 p.m. Weekly luncheon meetings at the Hotel Clark Coffee Shop, every Friday at 12 noon.

Metropolitan—Meeting in the auditorium of the Engineering Societies Building, January 21, at 7 p.m. The Junior Branch will meet January 14 and 28 in the ASCE Board Room, 33 West 39th St., at 7:30 p.m.

Michigan—Meeting in Detroit, Mich., January 12.

Nebraska—Meeting in Omaha, Nebr., January 21.

Oregon—Annual meeting at the Multnomah Hotel, Portland, Oreg., January 9.

Philadelphia—Meeting at the Engineers' Club, January 13. Meetings of the Junior Forum are held on the fourth Tuesday of each month except March.

Providence—Meetings are held in the Providence Engineering Society auditorium on the second Thursday of each month at 8 p.m.

Sacramento—Weekly luncheons at the Elks Temple every Tuesday, at 12 noon.

South Carolina—All-day meeting and banquet at the Columbia Hotel, Columbia, S.C., January 23, starting at 10 a.m.

Scheduled ASCE Conventions

SAN FRANCISCO CONVENTION
Fairmont Hotel
March 2–7,
1953

MIAMI BEACH CONVENTION
Casa Blanca Hotel
June 17–19,
1953

NEW YORK CONVENTION
Hotel Statler
October 19–23
1953

Notes from the Local Sections

(Copy for these columns must be received by the tenth of the month preceding date of publication)

Student Chapter members from the University of Akron swelled attendance at the regular October meeting of the **Akron Section.** Norman Green, president of the Chapter, said that he believed student participation will continue through the year. City Traffic Engineer Herb Woodling held the attention of all with a talk on interplay between engineering and politics.

The **Arizona Section** convened for two days, November 28 and 29, with the Arizona sections of seven other national engineering societies. A joint banquet, on the 28th, heard John D. Coleman, president of NSPE. At a business meeting Harold C. Schwalen was installed as president of the Section, Dario Travaini and Hanen H. Williams, vice-presidents, and Wilbur L. Heckler as secretary-treasurer. At a luncheon held jointly with AAE, James Barney received the Award of Recognition for achievement in early Arizona engineering. Included in the technical program were papers on "Need for More Highway Revenue," by Hanen H. Williams; "Current Phoenix Water Works Improvement Program," by William D. Williams; "Goodyear Farms Well," by Harold W. Yost; and "Organization and Program of Arizona Power Authority," by K. Sewell Wingfield, administrator of APA.

Members of the **Buffalo Section** opened their forum to debate the much discussed and internationally significant St. Lawrence Seaway. At a luncheon on November 18, Col. Philip R. Garges, district engineer for the Buffalo District of the Corps of Engineers, spoke on "The St. Lawrence Seaway and Power Development in the International Rapids Section."

At a joint meeting the **Central Illinois Section** and the Peoria chapter of the Illinois Society of Professional Engineers heard A. D. Spicer report on current developments toward unification of the profession. He noted that all but one of the members of the exploratory committee voted for adoption of Plan A, the EJC plan, and that the one dissenting vote favored Plan D (NSPE plan). The discussion further propounded the engineer's responsibility in the technological society of today. After dinner, Philip Zell Horton was presented with a life membership award.

Corrosion in action was the topic before the **Central Ohio Section** meeting held jointly with a meeting of the Columbus Technical Council. Frank L. LaQue, of the Development and Research Division of the International Nickel Co. Inc., and internationally known authority on the topic of corrosion, gave the talk of the evening, which was followed by Inco's outstanding color film on the subject.

Sanitary engineers held the floor at the **Cincinnati Section** meeting of November 5, with a panel of nine experts in the field discussing administration, education, design and research activities, state and municipal problems. Vernon G. MacKenzie, officer-in-charge, Environmental Health Center was moderator of the program. Participating in the panel were Edward J. Cleary, Cornelius Wandmacher, Vernon G. MacKenzie, Harlan Mace, Paul D. Haney, A. B. Backherms, Albert H. Stevenson, Charles D. Yaffee, and Hayse Black.

Forty members of the **Connecticut Section,** meeting at the Cafe Mellone in New Haven, heard George G. Hayden of the New York District Office of the Portland Cement Association, discuss "Soil Cement."

Changes in the Registration Law recommended by the Utah Engineering Council were discussed at the November 13 meeting of the **Intermountain Section.** The discussion of the model law was led by Le Roi Chadwicke, chairman of the Committee on Registration Law, ably assisted by F. O. Wold, secretary of the UEC. The following motions with regard to the law were unanimously carried: (1) That the definition of Engineer-in-Training be retained as presented in the Model Law; (2) that Section 3 of the Model Law be approved as is, with the exception of changes recommended by the UEC; (3) that the Intermountain Section of ASCE approve higher registration fees, provided the proceeds of the increased fees be utilized to police the Model Law; and (4) that the Intermountain Section approve Items 1, 2 and 3 of Section 12 relating to general requirements for registration as modified by the UEC.

Newly elected officers of the **Iowa Section** include: Philip F. Morgan, president; Neil Welden, vice-president; and L. O. Stewart, secretary-treasurer. At the first of two meetings held November 20, the presidents of the Student Chapters at the University of Iowa and Iowa State College described the work of their Chapters. "New Standard Bridges" for the Iowa Highway Commission was the contribution of P. F. Barnard, design engineer of the Commission, to the technical program. Prof. Ned Ashton of the University of Iowa completed the program with a discussion of "New or Unusual Structures in Northeastern U. S. and Canada." At a dinner meeting held subsequently, life memberships were awarded to Robert Caughey, William Schlick, Ernest Welden, Howard Green, and Cyrus Melick. Dean F. M. Dawson concluded the program with his observations as "An Engineer in Pakistan."

ASCE MEMBERSHIP AS OF DECEMBER 9, 1952

Members	8,263
Associate Members	10,407
Junior Members	16,903
Affiliates	67
Honorary Members	42
Total	35,682
(December 8, 1951	33,815)

Prof. Andre Jorisson, head of the hydraulics department of Cornell University, described for members of the **Ithaca Section,** at the November 19 dinner meeting, his recent visit to ten universities in Belgium, France, Switzerland, and Italy. Particularly noted in the discussion was the work being done by European hydraulic laboratories.

The controversial topic, "The Effect of Truck Loads on Pavements," was discussed before the **Kansas Section** by H. L. Aiken. A social hour preceded the evening business and technical activities.

Maryland Section members elected William B. Spencer, president; William R. Kahl, vice-president, and Richard Stephens, secretary-treasurer, at their annual meeting on December 10. Certificates of life membership were awarded to Gen. Donald H. Connolly, John Lansdale, Austin F. Shure, and George W. Stephens, Jr.

Well over 300 members of the **Metropolitan Section** gathered to hear J. O. Bickle and W. H. Bruce, Jr., of Parsons, Brinckerhoff, Hall and Macdonald discuss design and construction of precast tunnels. One of the interesting problems in the sinking of the Baytown Tunnel (one of the two discussed) was the presence of brackish water which changed buoyancy as the tunnel sections sank. The earlier meeting hour of 7 p.m. met with considerable approval.

After a series of field inspections and a lengthy study of building construction practices in the state of Florida, the **Miami Section** has recommended to the building officials in south Florida the adoption of a new inspection policy in the interests of public safety. The Section believes that by tightening up inspection of the construction of buildings and structures the quality of construction would be improved. The recommended policy would provide, among other things, for full-time inspection during the pouring of all concrete in structures, large and small.

At the **Michigan Section's** annual Christmas party and business meeting held in Detroit on December 16, Howard Peckworth, managing director of the American Concrete Pipe Association, spoke on "The ASCE—Looking Back and Ahead 100 Years." The following new officers for 1953 were installed: Dudley Newton, president; Benson J. Wood and John C. Kohl, vice-presidents; and Fred H. Burley, secretary-treasurer. A committee of Junior Members headed by Lloyd T. Cheney, of the Section arranged the meeting. An attractively printed "Michigan Section Newsletter" made its first appearance in December.

The part played by engineers in the rapidly expanding public health program of the U. S. Government was outlined to members of the **Northwestern Section** at its November 3 meeting. Herbert M. Bosch, professor of public health engineering at the University of Minnesota, noted the tremendous results accomplished by the

engineering and medical professions working together. E. G. Wagner, chief engineer and deputy chief of field party, Institute of Inter-American Affairs, Brazil, discussed the activities of his organization in developing health programs in Latin America.

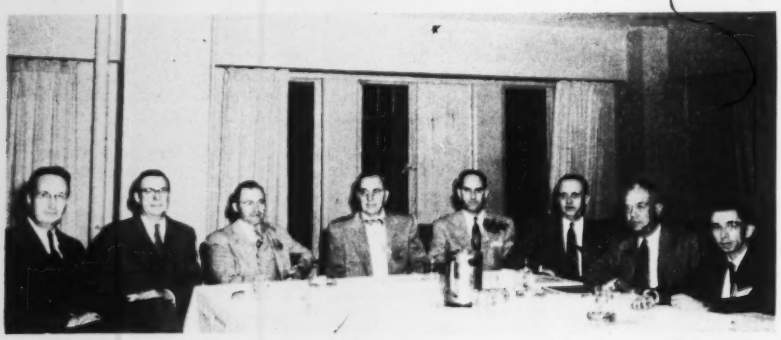
An enthusiastic audience of **Oregon Section** members heard H. Brian White of the British Columbia Engineering Co., describe the exciting Kemano project. Mr. White estimated that the plant would produce 500,000 tons of aluminum annually.

As a part of its community responsibility, the **Philadelphia Section** has undertaken and nearly finished a building-to-building survey of 50 blocks in central Philadelphia to determine what buildings are adequate for shelter against atomic attack, and what alterations in others would be necessary to convert them to shelters. "Is Construction a Backward Industry?" Not according to the speakers at the November 11 meeting of the Section held jointly with Philadelphia chapters of the AIA and the AGC. The speakers were J. Roy Carroll, Edward H. Cushman, J. Russell Cullen, and Joseph Hovell, representing architects, owners, contractors, and engineers, respectively. Approximately 400 engineers and guests of the **Delaware Sub-Section** gathered at the Hotel Du Pont in Wilmington to hear Dr. Alexander C. Monteith, vice-president in charge of engineering of the Westinghouse Electric Corp., emphasize the duty of the engineer to unify his profession and to foster the development of the young engineer. On November 12 members of the **Central Pennsylvania Sub-Section** visited the laboratories of Pennsylvania State College and then went on to visit the Garfield Water Tunnel.

Fall Meeting of Mid-South Section Marks Centennial

Dedicating its annual fall meeting—held in Memphis, Tenn., November 5–7, to the ASCE Centennial year, the Mid-South Section devoted a long session to discussion of "Centennial highlights" and the Chicago Convocation events, headed by E. F. Bespalow. At the banquet on November 6, a life membership certificate was presented to H. S. Gladfelter, and E. L. Chandler, Assistant Secretary of the Society, spoke on the proposed redistricting and rezoning plan. The principal speaker, Capt. Wallace B. Short, of the Sixth Naval District at Charleston, S.C., described the functions and activities of the Navy's Civil Engineer Corps. At the luncheon on November 7, Frederick H. Kellogg, dean of the School of Engineering at the University of Mississippi, discussed his recent work as consultant to the Punjab government on dam construction. The technical program covered such pertinent topics as "Modern Airport Pavements for Jet Planes" and "Industry and the Mid-South."

Of major interest during the business meeting was the presentation of a report prepared by a committee of Juniors, who were asked to find a remedy for lack of Junior interest in the Mid-South Section meetings. The committee, which was headed by Fred C. Walpole, suggested that: (1) Older members should make personal contact with Junior Members and encourage their more active participation in Society events; (2) the Society should appoint Junior Members to committees and other responsibilities; (3) meeting topics should be arranged to represent the interest of the majority of the members; (4) more time should be allowed on meeting programs for informal discussion of technical papers after formal presentation of the papers; and (5) the Society should encourage employers to partially subsidize attendance of Junior Members at out-of-town meetings.



Guidance of Mid-South Section affairs is largely in the hands of these members, photographed at Memphis. Shown, left to right, are F. T. Quinn, Jr., Memphis, director; L. T. Sumner, president, Memphis Branch; C. O. Wagner, Memphis, director; W. J. Turnbull, Vicksburg, past-president; B. H. Biggers, Jackson, Miss., director; L. A. Tvedt, Memphis, vice-president; K. W. Lefever, Little Rock, president; and W. G. Shockley, Vicksburg, secretary-treasurer.

Water resources, a topic of vital importance to most parts of the United States, was the concern of the **Providence Section** at its November 12 meeting. In a talk on "Water Resources of the State of Rhode Island," Arthur D. Weston, of Charles A. Maguire and Associates, covered public and industrial water supplies and water power in the state.

"Educational and entertaining" were the words used to describe a well attended meeting of the **Tacoma Section** on "Prestressed Concrete," held on November 11. The speaker of the evening was Arthur Anderson. New Section officers are A.M. Buell, president; H.J. Whitacre, vice-president; and G.H. Andrews, secretary-treasurer.

The **Fort Worth Branch** of the **Texas Section** heard Norman F. Strachan comment on the destructive effects of the Hiroshima and Nagasaki atom bombs at its December 8 meeting. Mr. Strachan, a Ft. Worth engineer, was a member of the U. S. Strategic Bomb Survey Team. In another part of the state, members of the **Houston Branch** heard the Houston district manager of the Raymond Concrete Pile Co., Herschel B. Miller, discuss cast-in-place concrete piles. The problems of welded structures were discussed by members of the San Antonio Branch with W.L. Perry of the Mosher Steel Co., acting as moderator.

At a meeting in Davenport, members of the **Tri-City Section** were addressed by ASCE Director L.H. Howson, who devoted considerable discussion to publication procedures of the Society. A. P. Rinell, L.A. Carlson, and Robert Erickson were elected, respectively, president, vice-president and secretary-treasurer.

Members of the **West Virginia Section** inspected and discussed the new Tri-State Airport. The speakers explained the difficult task confronting the planners when it was found that Kentucky and Ohio were not permitted to join in the financing. At a special ceremony, life memberships were presented to F.D. McEnteer and P.J. Walsh.

Opportunities for the engineer in civil service and public service were described for the benefit of members attending the December 4 meeting of the **Wisconsin Section** by Volmer H. Sorensen, director of the Wisconsin Bureau of Personnel, and George A. Sievers, industrial consultant and psychologist. Both urged a more aggressive attitude on the part of engineers in private practice to correct salary and job classification inequities in government service. New officers elected are Herbert O. Lord, president; Ralph E. Boeck and Ralph P. Larsen, vice-presidents; and Richard C. Dess, secretary-treasurer. The history, function, and purpose of trade associations were discussed at the November meeting by William Hart, local representative of the American Institute of Steel Construction. His talk also brought out some of the changes taking place in the design and use of steel, especially in regard to bolted versus riveted joints.

PENNSYLVANIA TURNPIKE EXTENSION

Topographic survey for 140-mile highway. Ground methods estimate: $415,000 and 2½ years. AERO delivered completed maps 145 days after start for $83,000.

Saving—$332,000 and 769 days.

BEAR MOUNTAIN HIGHWAY LINK

Topographic survey for 31-mile New York highway. Ground methods estimate: $74,400 and 2 years. AERO delivered maps in 90 days, for $17,500.

Saving—$56,800 and 640 days.

210,000-ACRE AEC FACILITY

Topographic survey for huge new Atomic Energy facility in S. Carolina. Maps for 210,000 acres were delivered 160 days after start.

Saving—years faster than ground surveys.

CITY MAPPING FOR DURHAM, N.C.

Topographic survey of 31-sq. mile city area. AERO mapped in 7 months for $30,000. Ground methods estimate: costs $180,000 and over 5 years.

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NEWS BRIEFS . . .

Rise in 1953 Construction Activity Forecast

New construction activity in 1953 is expected to reach a new peak, with a probable increase of about $1 billion over the $32¼ billion apparent for 1952, according to outlook estimates prepared jointly by the U.S. Labor Department's Bureau of Labor Statistics and the Building Materials Division of the U.S. Department of Commerce. Some expansion is likely in the coming year in both private and public construction.

The forecast for record construction activity in 1953 is based on the assumption that business will remain good, buoyed in part by increasing defense expenditures, at least in the early part of the year. It is also assumed that, barring major international developments, materials will be plentiful, that the limited controls in force will not interrupt construction operations, that the labor supply will be adequate, and that costs will remain relatively stable.

A peak of $22.2 billion in expenditures for new private construction is anticipated, in the expectation of a continuing high level of housing activity and expansion in public utility plant and commercial building. It is expected that about as many private dwelling units will be started during the year as in 1952, when the million-unit mark will again be exceeded. Expenditures for new private housing actually put in place, however, will (at $10.2 billion) be somewhat greater than in 1952, because the year will begin with a greater number of units already under way. Prediction of an active home-building year is based primarily on the expectation of favorable economic conditions and ready availability of mortgage funds.

Next year will probably be the tenth successive year of increasing construction activity by the public utilities. Announced expansion goals suggest unprecedented expenditures of about $4⅓ billion, with most of the 11 percent rise in volume over 1952 occurring in the gas and electric light and power group for which there is an extensive backlog of both industrial and domestic need.

Commercial building activity will probably increase more than 25 percent in 1953, continuing the recovery of recent months resulting from improved materials availability and removal of credit controls. On the other hand, private industrial building is expected to drop by about the same proportion—from the 1952 record outlay of about $2¼ billion, as defense plant expansion programs approach completion.

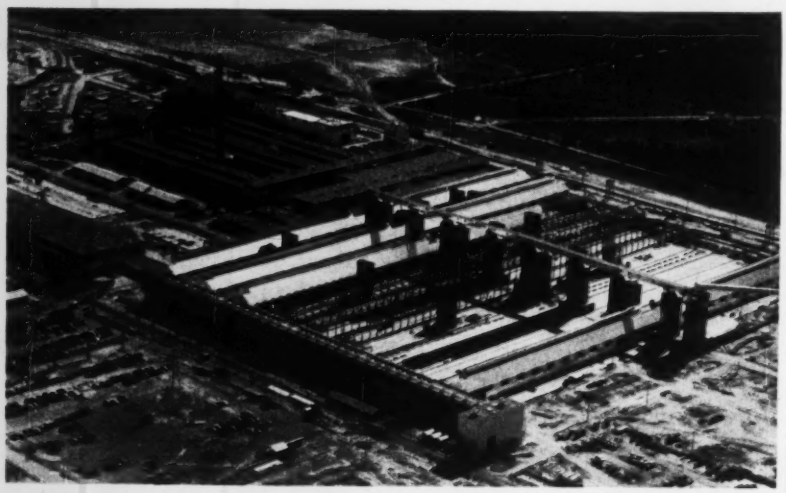
Although 1953 will be another active defense year, the anticipated rise in total public expenditures for new construction, from $10.6 to $11.3 billion, will probably result as much from increasing activity in civilian as in military types of public work. Highway construction will probably increase nearly 10 percent to a new high of $3 billion, reflecting the expanding program of federal aid to highway building, and an anticipated large volume of state toll-road construction. Ground has already been broken for the $300 million Ohio Turnpike project, and work is proceeding rapidly on the $500 million New York State Thruway.

Public school building will continue its postwar expansion into 1953 in response to constantly increasing classroom needs. Expected expenditures of about $1.8 billion for schools represent a 10 percent increase over the 1952 total, and the largest amount of classroom space ever to be put in place in a single year.

According to the joint forecast, outlays for military and naval facilities will probably rise about 20 percent to around $1.6 billion, as considerable new work (much of it already under contract), gets under way and the extensive current program is completed. A much more moderate increase (3 percent) will occur in public industrial construction, resulting entirely from additions to existing atomic energy facilities and commencement of the hugh atomic plant near Portsmouth, Ohio. Construction of industrial facilities for the Army, Navy, and Air Force, mostly the rehabilitation of existing ordnance plants, will be substantially completed during 1953.

Public residential building will drop from this year's level, reflecting the tightened statutory limitation on the start of new federally subsidized housing units. Virtual completion of the Veterans Administration program of hospital construction and reduced funds for the program of federal aid to state and local hospitals will cause a substantial decline in the rate of publicly aided hospital building.

Aluminum Production Begins at Rockdale, Tex., Plant



Nation's newest aluminum smelting plant—Rockdale (Tex.) works of Aluminum Company of America—nears completion. With capacity of 85,000 tons of aluminum a year, plant will reduce refined bauxite ore (alumina) into metal through an electrolytic process. Completed project will include smelting plant with four potlines; plant for the manufacture of carbon electrodes required in the electrolytic aluminum-producing process; power plant for the generation of required electricity; and facilities for mining lignite. Aluminum was poured from first operating potline on November 24, and second potline was ready for operation in December. Third and fourth units are scheduled for completion next spring, but will not be put in operation until fall when the new lignite-burning power plant is ready for operation. Pending completion of the power plant, electric power for the first two units will be supplied by Texas Power & Light Co., which will operate the power plant for ALCOA. Power plant is being designed and built by Ebasco Services, Inc. Construction of the Rockdale works, first aluminum smelting plant to use electric power generated by burning lignite, was begun in October 1951. Fabrication and erection of the 11,000 tons of structural steel required for the project was handled by the Bethlehem Steel Co.

Engineers Chosen for Garden State Parkway

Further progress has been made by the New Jersey Highway Authority in meeting the three-year target date set for completing the $285,000,000 Garden State Parkway, with the award of engineering contracts for a number of the thirteen construction sections into which the project has been divided. Sections 5, 6, 9 and 12 are already completed.

The engineering contract for Section 2 is held jointly by Fay, Spofford & Thorndike, of Boston, and Madigan-Hyland, of Long Island City, N.Y. Madigan-Hyland also has the contract for Section 3.

Engineers selected for Section 4 are Dr. Albert C. Jones, of Mt. Holly, N.J.; Dr. E. Lionel Pavlo of New York; Parsons, Brinckerhoff, Hall & Macdonald, of New York; the New Jersey State Highway Department; and Grassman & Kreh, of Hillside, N.J.

The contract for Section 7 is held jointly by Ammann & Whitney, of New York; Edwards, Kelcey & Beck, of Newark, N.J.; Frank E. Harley, of Matawan, N.J.; Frederic R. Harris, Inc., of New York; and D.B. Steinman, of New York.

Porter-Urquhart Associated, consulting engineers of Newark, N.J., have the engineering contract for Section 8.

Engineers for Section 10 are Sherman, Taylor & Sleeper (Camden, N.J.) and A.C. Jones Associated; Dr. E. Lionel Pavlo, D.B. Steinman, and Hardesty & Hanover, of New York; and Brown, Blauvelt & McFarland, Inc., of Woodbury, N.J.

Engineers for Section 11 are Gannett Fleming, Corddry & Carpenter, Inc., of Harrisburg, Pa., and the J.E. Greiner Co., of Baltimore, Md.

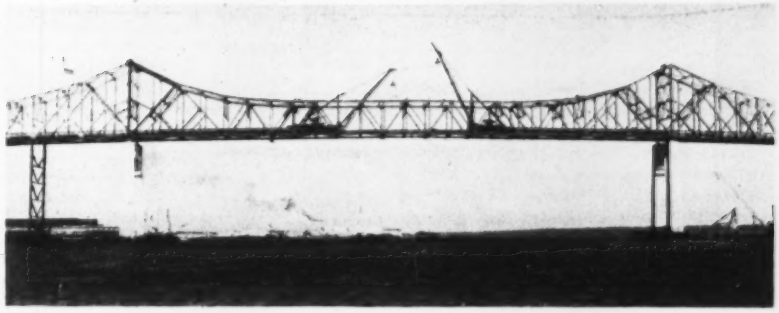
Parsons, Brinckerhoff, Hall & Macdonald have the over-all engineering contract for the 180-mile route.

Link-Belt Expands Its Production Facilities

As a step in expanding its production of conveying and processing equipment, the Link-Belt Co. has built a new plant, its seventeenth, at Colmar, Pa. The main manufacturing building is 300 by 880 ft. The structure is divided into five longitudinal bays, four of which have overhead cranes. Transverse bays at either end, also equipped with overhead cranes, will handle the receiving and shipping of supplies. Engineering of the diversified products of the plant will be done in a two-story 100 by 240-ft brick structure immediately adjacent to the manufacturing building.

Both structures are steel frame with roof trusses spanning the 60-ft bays. The exterior of the manufacturing building is corrugated asbestos siding, and of the office building common brick. The plant has its own water supply and sewage treatment plant. It was built by the Austin Co.

St. John's River Bridge in Florida Nears Completion



Erection of bottom chord member that ties together cantilevered closing spans in new John E. Matthews Bridge, spanning St. John's River between Jacksonville and Arlington, Fla., has just been completed in this view of structure, first high-level bridge in Florida. Part of Jacksonville Expressway Project, four-lane 7,375-ft structure consists of 3,447-ft, eastern approach to main span, 2,306-ft western approach, and 1,622-ft main span over Terminal Channel. Main span is made up of two anchor spans, each 406 ft long, and central 810-ft span. Bridge clearance is 149½ ft above river at mean sea level. Built for Florida State Road Department, project was designed by Associated Engineers and Architects, consisting of Smith, Reynolds & Hills of Jacksonville, and Parsons, Brinckerhoff, Hall & Macdonald, of New York. Job of fabricating and erecting 13,000 tons of steel required in superstructure was handled by Bethlehem Steel Co. and Merritt-Chapman & Scott were contractors on substructure.

Fairless Plant of U.S. Steel Begins Production

Operation of a huge new steel plant—the $450,000,000 Fairless Works of the United States Steel Corp. at Morrisville, Pa.—began on December 11 with the lighting of the first of its two blast furnaces and the tapping of an open-hearth furnace. The second blast furnace will be ready for operation in January, by which time others of the plant's nine open-hearth furnaces will also be in production.

The two blast furnaces will have an annual output of 1,200,000 tons of iron, which will be melted in the open-hearth furnaces to produce 1,800,000 tons of steel ingots each year. Work is progressing rapidly on other facilities that will have an annual productive capacity of 235,000 tons of hot rolled sheets, 289,000 tons of cold rolled sheets, 170,000 tons of tin mill products, 285,000 tons of bar products, and 280,000 tons of small-diameter pipe. Production of these materials is expected in the fall of 1953.

The lighting up ceremonies were conducted by Nancy and Carol Fairless, young granddaughters of Benjamin Fairless, M. ASCE, chairman of the board and president of United States Steel, for whom the Fairless Works is named. In keeping with a tradition that blast furnaces be named for their lady sponsors, the two furnaces placed in operation were named for the little girls, and the No. 2 blast furnace was christened "Hazel" in honor of Mrs. Benjamin F. Fairless. Mr. Fairless hailed the project as, "above all else, the product of that unique American way of life, which it is designed to preserve."

Built in less than two years on a 6-sq mile tract on the Delaware river south of Trenton, the Fairless Works constitutes the company's largest addition to its production capacity since the beginning of the defense emergency. The huge mill has 75 miles of standard-gauge railroad track, 20 miles of improved roads, 30 miles of sewer, a water-treatment plant with a capacity of 254 mgd, and several miles of belt conveyors. It has two batteries of 87 coke ovens each, one of which has just made its first coke. Although the plant is now obtaining its electricity from the Philadelphia Electric Co., it will ultimately operate two 30,000-kw generators of its own.

Benjamin Fairless, M. ASCE, chairman of board of United States Steel (left), watches as his granddaughter touches oil torch to a fuse that fires the No. 1 blast furnace put in operation at new Morrisville, Pa., mill. At right is Albert Berdis, who will manage mill.

Atomic Power and World Fuel Reserves

The question is often asked, "How soon should we get serious about useful power from atomic energy on a large scale?" An answer was given by Dr. L. R. Hafstad, Director, Reactor Development, Atomic Energy Commission, in an address before the 57th annual Congress of American Industry, sponsored by the National Association of Manufacturers in New York in December. The estimates, in respect to fuel consumption, demand, and reserves, in the following quotations are taken from a study made for AEC by Palmer C. Putnam. "If you take a look at the total world energy demands," Dr. Hafstad said, "you are examining very big numbers, of course; so the study uses a new quantity, the "Q," just the letter Q. One Q is 10^{18} Btu, a million, million, million British thermal units. For the year 1947, according to the survey, the energy burned up in all the world for all purposes was only $^1/_{10}$ of a Q.

"Now," he continued, "we want to see what the energy demands of the world are likely to be in the future. First take a look at the rate of increase of the world's population and at the rate of increase in the per capita energy demand. The per capita demand for energy is going up by leaps and bounds, and so is population all over the world. If you put those two numbers together, then you will find that the energy demand is rising exponentially, and you will find that as we extrapolate into the future it becomes apparent that before very long there is going to be a squeeze for energy—if, of course, the increases continue.

"Historically, the picture this study draws looks something like this. Between the year 1 A.D. and the year 1860 when the Industrial Revolution really got under way, the total energy consumption for the whole world totaled roughly 6 to 9 Q. From 1860 to 1947, the world burned about 4 Q. Already, the annual rate of consumption is equivalent to 20 Q per century, and by the year 2000 will be about 100 Q per century. This gives you a bird's-eye view of what the energy demand is going to be, assuming present industrial trends continue.

"Now let's take a look at the reserves, on which estimates vary considerably. The best figures that the study could settle upon, based on data from the U.S. Geological Survey and other sources, is that the world's total coal and oil energy reserves are somewhere in the neighborhood of 36 Q. This would be less than 100 years' supply at the rate of demand of the next generation or two. Of course, coal is by far the largest fossil fuel energy reserve, accounting for 30 of the 36 Q. Oil and gas reserves are estimated at about 6 Q.

"Now, the question is, with this demand for energy and the relatively small reserves, should we go directly to solar energy, as President Conant of Harvard suggested last year, or has atomic energy something to contribute, at least in the interim period while we are learning how to use solar energy effectively?

"A great amount of uranium is available, but only some very much smaller amount is economically recoverable. On the basis of the best guesses from the information that can be obtained, the survey's figures turn out to be of the order of 100 to 1,000 Q for the energy of the world's uranium reserves, depending on the value put on the metal and the extent of breeding accomplished. Thus we would have several centuries in which to use atomic energy as a supplement to the energy reserves of coal and oil, during which time the scientists and engineers of the world could learn how to use solar energy.

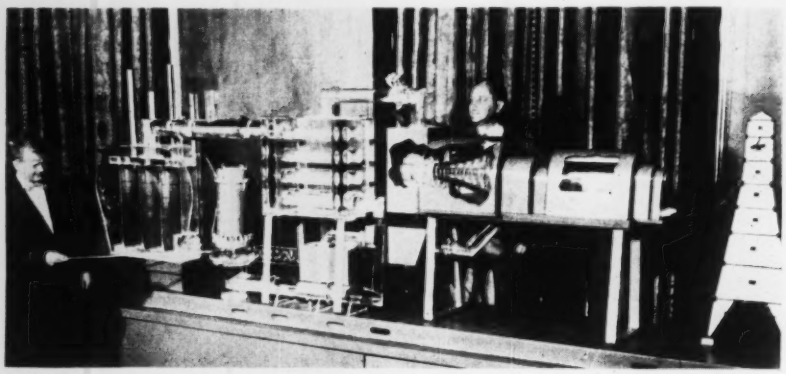
"In short," he concluded, "there seem to be adequate reserves of uranium. Costs now are high, but in a new field, such as atomic energy, the costs undoubtedly will decrease. Costs will be brought down partly by the technology we are getting from the developments on military mobile reactors where economic considerations are not so critical. As costs come down in the atomic energy field, they will be rising in the competitive energy fields of coal and oil. At some point in the future, these curves will cross, and at that point we will move into the genuine era of atomic energy—not before that time."

Atomic Power Plant for "Nautilus" Nearly Completed

A model of an atomic power plant built by the Westinghouse Atomic Power Division was given its first public showing in New York in December by Charles H. Weaver, manager of the Division, at the 57th annual Congress of American Industry, sponsored by the National Association of Manufacturers.

He reported events that are furthering progress on the company's Submarine Thermal Reactor project. First, a land-based prototype submarine plant, realistic even to the extent of being surrounded by a large tank of water simulating the sea, was erected in the middle of the Idaho desert. Second, at Groton, Conn., in June 1952 the keel of the submarine *Nautilus SS- 571*, the country's first atomic-powered ship, was laid. For this project Westinghouse is building a second nuclear power plant. The Electric Boat Division of General Dynamics Corp., is working as a subcontractor on the project.

On November 14, Westinghouse announced that the Newport News Shipbuilding & Drydock Company has joined them as subcontractor, in the additional responsibility of designing and developing a nuclear plant suitable for propelling aircraft carriers.



Charles H. Weaver (left), manager of the Westinghouse Atomic Power Division, and W. E. Shoupp, director of development, display model atomic power plant at the 57th annual Congress of American Industry. Columns at extreme left end of model represent nuclear reactor or atomic pile which generates heat from splitting atoms. Control rods extend from top of reactor. Water or some other heat transfer fluid circulates through heat exchanger toward center of unit. Steam generated there drives turbine and electric generator at right. Electricity is carried to a transmission tower. Mr. Weaver is pointing to a pump that circulates water from the reactor to the heat exchanger and back to the reactor again. In a submarine the turbine would drive the propeller shaft. Westinghouse photo.

W. E. Robertson, Pioneer Engineer, Dies in Texas

William E. Robertson, pioneer Texas engineer and industrialist, died at his home at El Paso, Tex., on November 10, at the age of 86. Mr. Robertson founded the El Paso Bridge and Iron Co. and was president of the Virginia Bridge & Iron Co., now a subsidiary of the U.S. Steel Corp. He was widely known as an authority on water supply, and the million-dollar El Paso water treatment plant was named in his honor in recognition of his services to the city.

Ventura County, Calif., Plans Water Conservation Project

The Board of Directors of the United Water Conservation District of Ventura County, California, has adopted a report of its general manager and chief engineer, Julian Hinds, M. ASCE, calling for construction of two storage dams on the Santa Clara River system and for canals and pipelines for diverting water to irrigated lands and to spreading grounds for storage underground. The $18,500,000 project includes construction of a concrete arch dam, about 300 ft high, at the Topatopa site on Sespe Creek to store 50,000 acre-ft; a 200-ft-high rolled-fill dam at the Santa Felicia site on Piru Creek for storage of another 100,000 acre-ft; and a conduit system for conveying water to spreading grounds near El Rio and Saticoy.

Although average rainfall in the area is about 17 in. a year, it varies greatly from year to year. During floods there is great wastage to the Pacific Ocean. It is planned to increase the 78,000 highly productive acres, now under cultivation, to 143,000 acres. In the past, heavy drafts of irrigation water from wells have reduced ground-water levels to as much as 70 ft below sea level and several wells have become salt—a situation in urgent need of correction.

The Board of Directors of the District proposes to finance construction of the conservation works by issuing bonds after approval by the voters of the District is obtained.

Engineers Warned of "Spanish Prisoner" Swindle

The hopeful crooks now operating the hoary and time-worn "Spanish Prisoner" swindle from Mexico apparently have acquired an old copy of the ASCE Official Roster. Reports from many parts of the country indicate that members are receiving the following unsigned, airmail letter crudely typed. In all cases, members are carefully addressed as "Dear Engr. ——."

"Dear Engr. ——:

"A person who knows you and who has spoken very highly about you, has made me trust you a very delicate matter of which depends the entire future of my dear daughter, as well as my very existence. I am in prison sentenced for bankruptcy, and I wish to know if you are willing to help me to save the sum of $450,000 U.S. Cy., which I have in bank bills hidden in a secret compartment of a trunk that is now deposited in a customhouse in the United States.

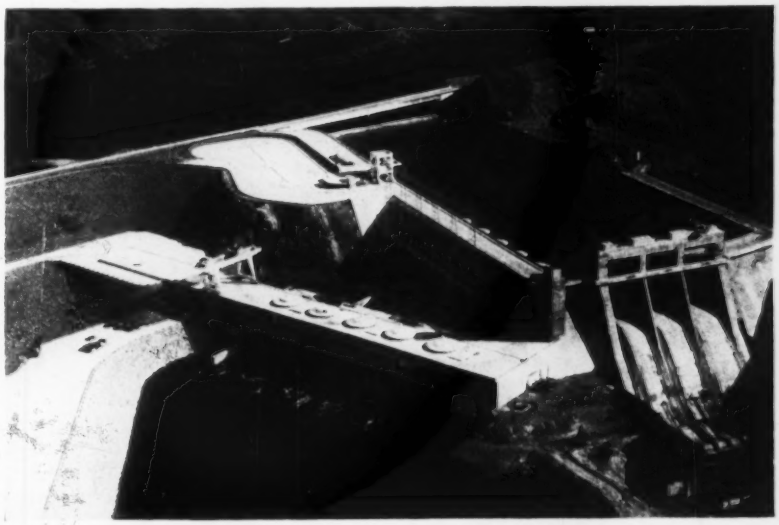
"As soon as I send you undeniable evidence, it is necessary that you come here and pay the expenses incurred in connection with my process, so the embargo on my suitcases can be lifted, one of which suitcases contains a baggage check that was given to me at the time of checking my trunk for North America, and which trunk contains the sum above said. To compensate you for all your troubles, I will give you the THIRD PART OF SAID SUM. Due to serious reasons which you will know later, please reply via AIR MAIL. I beg you to treat this matter with the utmost reserve and discretion. Fearing that this letter might have gone astray and not reach your hands, I will not sign my name until I hear from you, and then I will entrust you with all my secret. For the time being I am only signing 'F.'

"Due to the fact that I am in charge of the prison school, I can write you like this and entirely at liberty. I cannot receive your reply directly in this prison, so in case you accept my proposition, please AIRMAIL your letter to a person of my entire trust who will deliver it to me safely and rapidly. This is his name and address:

Sr. Antonio Rios
Monterrey 122-1
Mexico 7, D. F."

Although CIVIL ENGINEERING policy is to avoid stories solely and intentionally humorous we feel that this undue attention to engineers warrants publication. Any member receiving a letter of the type quoted should, of course, turn it over promptly to his local postmaster.

Davis Dam and Power Plant Dedicated



Completion of Davis Dam and power plant—$119,000,000 project on the Colorado River between Arizona and Nevada just north of their boundary junction with California—was celebrated by the Bureau of Reclamation in dedication ceremonies on December 10. Seventh in a system of dams begun with, and based upon, Hoover Dam 67 miles upstream, the present project completes a large-scale program for development of the Colorado River begun twenty years ago. Construction of the project was started in 1942, interrupted by the war, and resumed in 1946. The dam began storing water in January 1950, and power generation started a year later. The Davis power plant has five 45,000-kw semi-outdoor-type generators, the covers of which can be seen along the top of the powerhouse. Davis Dam was named after the late Arthur Powell Davis, Past-President and Honorary Member of ASCE and Commissioner of Reclamation from 1914 to 1923, whose widow (shown in upper view with E. G. Nielsen, M. ASCE, regional director of the Bureau) was honored in the dedication ceremonies. Bureau of Reclamation photos.

"Who's Who in Engineering" to Be Revised

Plans for beginning compilation of a seventh edition of *Who's Who in Engineering*, the first revision since 1948, were made at a recent meeting of the editor and publisher with the advisory Committee of Engineers Joint Council on the publication.

The method of revision to be followed includes resubmission of the biographies in the current edition to the engineers concerned, who will be asked to bring their records up to date. Invitations to supply data will also be sent to a long list of engineers who have been recommended for inclusion in the volume in the four years since the appearance of the sixth edition; to engineers not now included, whose activities are indicative of the quality of their professional accomplishments; and to engineers recommended by institutions and organizations with which they have been identified.

Minimum qualifications for inclusion determined at the recent meeting are:

1. Ten years' active engineering practice—at least five of them in responsible charge of engineering work.

2. Ten years of engineering teaching in colleges or schools of accepted standing, with at least five years in responsible charge of a major engineering course in the school or college.

Engineers with military assignments will be given proper consideration in both of these groups.

A publication of importance to both the profession and the country at large, the revised volume will be produced under careful editorship. The publisher—the Lewis Historical Publishing Co., County Trust Building, 8th Avenue at 14th Street, New York 11, N.Y.—will have the cooperation of professional groups and eleven engineering societies including ASCE.

New Compaction Process Permits Building on Sand

Tremendous construction possibilities are opened up by development of a new sand-compaction process called Vibroflotation that has permitted construction of a $12,000,000 phosphate chemicals plant for the International Minerals and Chemical Corp. of Chicago on dry wasteland sands at Bonnie, Fla. Savings of $250,000 were achieved on the project, the first large-scale plant in the United States to be built on sand without other support. The Rust Engineering Co. of Pittsburgh, builder of the Bonnie Phosphate Chemical Plant, has been given a franchise to handle vibroflotation by its inventor, Sergey Steuerman, M. ASCE, of New York.

The foundation problem at the site chosen was especially difficult since the soil must not only support heavy structures but also take shock and vibration without settlement, which would be highly damaging to the process buildings with their maze of piping and chemical apparatus. Soil studies showed that the site was underlain with an uncertain base of loose sand, a "sandy matrix stratum," and a third layer of compressible sand and gravelly clay, to a depth of some 60 ft before a suitable bearing stratum was reached.

The Vibroflotation process increases the relative density of sandy soil by means of a shaking and pushing process that packs the sand grains more closely and tightly together, reducing the voids or empty spaces between the particles. A much more solid sand mass results.

The device used, which is called a Vibroflot, consists of a tube vibrated by an electrically driven eccentric inside it and producing a ten-ton centrifugal force. This apparatus is attached to a follow-up pipe which houses required water and electric lines. In operation the Vibroflot is suspended from a crane and guided by vertical wooden leads. Vibrating at full speed, it is lowered into the sand while a water jet at its tip forms a saturated sand mass or temporary "quick sand" condition, into which the vibrator rapidly sinks.

Called a Vibroflot, the device used in the sand-compaction method consists of a tube vibrated by an electrically driven eccentric inside it and producing a ten-ton centrifugal force. In operation, the Vibroflot is suspended from a crane and guided by vertical wooden leads.

Lowered to an average depth of 13 ft at the Bonnie project, the device by its vibrating action pounds the sand surrounding it into a tighter mass on all sides. Fresh sand is shoveled in from above to fill the extra space emptied by this compaction. The vibrator is withdrawn in 1-ft stages, being run at each stage until the sand is packed as solidly as practical at each stage. An average of $2\frac{1}{2}$ cu yd of fresh additional sand is added by the time it is fully withdrawn. The result is a tightly compacted sand column about 8 to 10 ft in dia. The columns are pounded in pre-determined overlapping pattern with about 8 ft between centers. The pattern greatly strengthens the compaction results which give a relative density of 70 to 100 percent—more than enough for high or very high bearing capacity.

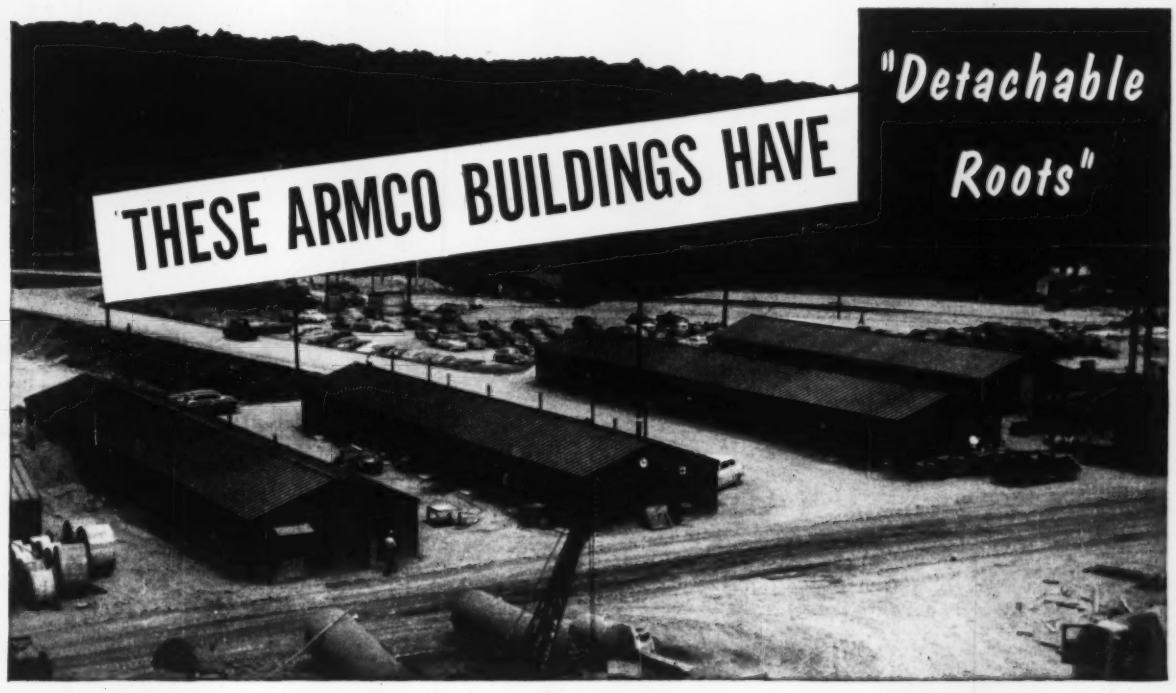
The complete compaction at the Bonnie plant took two crews 160 days for some 3,350 compactions. Results show, after six months, that one of the first structures completed, the phosphate storage building, has experienced no measureable settlement. Advance estimates had allowed for a maximum settlement of about 1 in.

FCDA Schedules West Coast Engineering Symposium

Instruction in the engineering essentials of defense in this atomic age will be given in a comprehensive three-day program arranged for West Coast engineers by the Berkeley (Calif.) Regional Office of the Federal Civil Defense Administration. The three-day symposium will be held at the FCDA Western Technical Training School, Saint Mary's College, Calif., February 5–7, with courses conducted by visiting technical officials of the FCDA, the resident faculty of the Western Technical Training School, and the Office of California Civil Defense.

Subjects scheduled for discussion on Thursday, February 5, are: The Architect and Engineer in Civil Defense; Restoration of Facilities; Streets and Highways; Utilities; Urban Analysis; and Engineering Equipment Stockpile. The Friday program will consist of talks on the following subjects: Emergency Plans; A Shelter Survey; Building Code Planning; Design of Structures in the Atomic Age; Windowless Structures; and Lateral Stability and Blast Analysis. Saturday morning will be devoted to rescue and engineering equipment tests and to review and evaluation.

Paul J. Prout, A.M. ASCE, general engineer for the Berkeley Regional Office of the FCDA, has been in charge of preparing the technical program. He is assisted by a committee composed of Ralph Tudor, representing ASCE; Lester Hurd and Chester Treichel, AIA; John Gould, SEA, Clyde Bentley, ASHVE; and Gen. Dwight Johns, SAME. Additional information may be obtained from local Civil Defense offices or Dr. B. F. Gillette, director of the Western Technical Training School.



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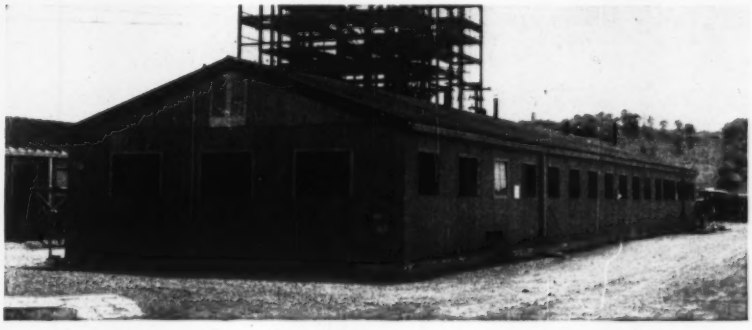
But what happens when a building has served its purpose in one location? That's easy if it is an Armco Steel Building. It can be completely dismantled without loss of material, moved and re-erected at a new site. Extending or rearranging the building is also a simple matter.

Next time you need dependable, low-cost shelter, try Armco Buildings for field offices, shops, storage buildings and warehouses. Write us for complete details. Armco Drainage & Metal Products, Inc., 1303 Curtis Street, Middletown, Ohio. Subsidiary of Armco Steel Corporation. Export: The Armco International Corporation.



Armco Building at construction job.

Hook bolts hold panels securely to base angle.

ARMCO STEEL BUILDINGS

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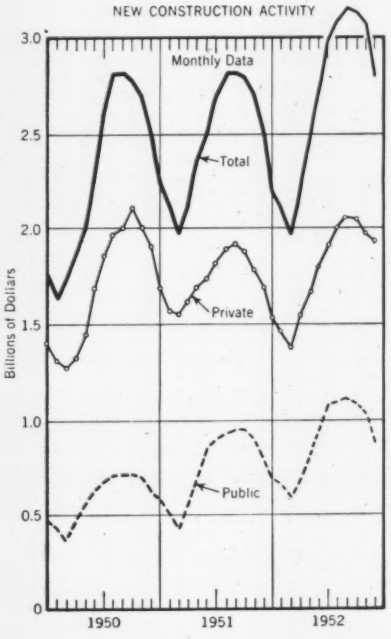
Decline in Construction Volume in November Noted

A 7 percent decline in expenditures for new construction in November, to a total of about $2.8 billion, is reported jointly by the U.S. Labor Department's Bureau of Labor Statistics and the Building Materials Division of the U.S. Department of Commerce.

All classes of construction activity, except commercial building, shared in the drop from October, although the decline was less than seasonal in most major types of work. Public construction dropped relatively more than private during the month because of a $100,000,000 decline in highway work which lagged slightly more than is usual for the time of year. However, the total dollar outlay for new public construction during the month was 9 percent above the November 1951 level, and private expenditures were 5 percent greater. Expenditures for private construction came to $1,917 million, and for public construction to $882 million.

For the fifth consecutive month, expenditures for private non-farm housing remained above the billion-dollar level, and commercial construction continued the slow but steady climb that began in April. The greatest October-to-November drop among major groups in the private section was in public utility construction—attributable to the regular fall decline.

The joint agencies note that new construction expenditures for the first eleven months of the year totaled $29,828 million—an increase of about 5 percent over the amount for the same period in 1951. The dollar volume of private work was about the same as last year (around $20 billion), but public outlays of $9,839 million were 16 percent higher than in the comparable period of 1951.



Seasonal drop of 7 percent in November construction activity, to total of almost $2.8 billion, is indicated in Department of Commerce curves.

Aid in Saline Water Research Program Asked

Individuals and organizations interested in the conversion of sea water and the demineralization of brackish water for beneficial uses are asked to participate in the first phase of a program instituted by the Office of Saline Water Research Coordination of the Department of the Interior. The objective of the program, which was authorized by the 82nd Congress, is to coordinate and stimulate research and development of economically feasible processes for making saline water of all kinds suitable for municipal, industrial, irrigation, and livestock uses. The first phases of the program consist of inventorying past developments, both theoretical and practical, and establishing contact with persons having something to contribute from personal experience in the field. The program will be headed by Goodrich W. Lineweaver, Assistant Commissioner of Reclamation.

A preliminary brochure outlining the work thus far accomplished and possible new areas of research is available. Inquiries should be addressed to the Saline Water Program, Department of the Interior, Washington 25, D.C.

Neare's N.G.* COLUMN

R. Robinson Rowe, M. ASCE

"Happy New Year," beamed the Professor.

"I hope you mean it," growled Joe Kerr. "You know, I'm still mad at your dig that I'd have to use tin-can models to solve your Operation 2-Tank II."

"If I guessed wrong, Joe, I'll buy."

"Let's bet both ways, Professor Neare, but I warn you that Cal Klater saw my answer and admitted it was practically the same as his. I started with his third equation:

$$adh/AdH = b\sqrt{h}/B\sqrt{H} = 1. \quad (3)$$

Again letting $h = u^2H$ and substituting the new data, $A = 2.5$, $B = 5$, $a = 2$, $b = 4$, it reduced to:

$$-dH/H = 2udu/(u^2 - u + 1.25) \quad (4)$$

I'll bet you thot I'd quit when I couldn't factor that quadratic, but I used Peirce's Tables to integrate it, finding,

$$-\log H = \log(u^2 - u + 1.25) + \arctan(u - {}^1/_2) + C \quad (5)$$

From the initial conditions, $H = 148.84$ and $h = 33.64$, I found $C = -\log 145.3124$ and evolved the horrendously implicit relation:

$$\tan \log \frac{145.3124}{h - \sqrt{hH} + 1.25H} - \sqrt{\frac{h}{H}} = {}^1/_2 \quad (6)$$

Now you asked for H when $h = 29.16$, but did you realize what you were asking for? Cut-and-try, cut-and-try, cut-and-try, ad infinitum, ad nauseam, ad-tut-and-cry. The thrill over finding $H = 154.47$ after 7 hr of trial was short-lived, for I realized that H must decrease from 148.84. Trying in the other direction, I found $H = 0.009$ would fit if I used $\sqrt{H} = -0.095$; why the minus sign I don't know, but anyway, H came out positive. Do I win?"

"I'd give him an E for Effort, Noah," proposed Cal Klater, "but his answer is more than 100% wrong, even tho it's practically the same as mine. I found it simpler to solve Eq. (4) in the parametric form:

$$H = Ce^m \sin^2 m$$
$$h = Ce^m(\cos m + {}^1/_2 \sin m)^2 \quad (7)$$

where $C = 145.3124e^{-\pi/2} = 30.2076$ and m is a parameter varying from 1.59538 at the start to zero when the upper tank is empty. At that stage, h in the lower tank is 30.2076, so when h drops to 29.16, the upper tank can't be any emptier. The answer is just exactly zero. Joe's answer was a redundant from the equation's attempt to discharge imaginary water from an empty tank under an imaginary head."

"Well Cal's right, but I'm going to give Joe a D for Diligence, or C for Confidence, or maybe a B for Both. Then, to start the new year with something easier and less academic, I'll introduce you to a new model of literal arithmetic. It looks like these samples:

VE×ER = ROWE E×GULP = PLUG
BO×ER = BAER FO×FUN = ONION
E×TIME = EMIT TE×ANS = PROUD

Each sample is an example in multiplication, with letters indicating individuality or repetition of digits, and with the times sign 2-timing as an X. Solutions are not necessarily unique; any one will do, except I'd like the largest product for the first one. Also, I'll enjoy other similar samples."

[*Cal Klaters were: Stoop (John L.) Nagle, David M. Rockwood, Tom G. Ogburn III, Rudolph W. Meyer, and Sauer Doe (Marvin Larson). Also Kum Pewter (Walter Steinbruch) solved the October Tooterville toots from a hospital bed.*]

How Hallandale, Florida, got Soft, Clear Water

PROBLEM

Well water with hardness of over 240 ppm--mostly bicarbonate--and objectionable organic color. Softening and color removal definitely required. Water Conditioning equipment to be economical in serving today's 5,000 population ... to retain this economy in providing 1980's population with 2,000,000 gpd.

SOLUTION

Briley, Wild & Associates, Consulting Engineers, Daytona Beach, Fla., studied latest methods & equipment. Selected lime-alum method. Specified modern type unit for highly efficient reaction between minerals in water and lime-alum. Installed--Permutit Vertical Precipitator. Upward filtration through suspended sludge blanket gives intimate chemical contact, makes for operating economy.

RESULTS

Softened, clear water. Permutit Precipitator removes color and reduces hardness to desired level. Last traces of turbidity are removed by filtration through gravity filters.

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DECEASED

Webster L. Benham, Former Director, Dies

Webster Lance Benham (M. '14) founder and president of the Benham Engineering Co., of Oklahoma City, Okla., died at his home on November 22, at the age of 70. Colonel Benham, who served in World War I, graduated from Columbia University in 1905, with the degree of C.E. Before organizing the Benham Engineering Co., in 1909, he had been a member of the firm of Benham & Adams, acted as assistant city engineer for Oklahoma City, and was an instructor in civil engineering at Epworth University. From 1919 to 1922 he was active in the consulting firm of Johnson & Benham, of New York and Kansas City. Colonel Benham was consultant to over 250 cities, including Dallas, Tex., New York, and Kansas City, and was the architect-engineer on construction of Camps Livingston, Beauregard and Polk during the last war. His many services to the Society included a term as Director, 1948–1951.

Webster L. Benham

Louis Philip Blum (M. '10) retired engineer of Pittsburgh, Pa., died there on November 4 at the age of 79. A member of Blum, Weldin & Co., civil and mining engineering firm of Pittsburgh, from 1916 to 1945, Mr. Blum was associated briefly with the Chester Engineers before his retirement. Previously he had been with the Philadelphia Bureau of Surveys for ten years and the W. G. Wilkins Co., of Pittsburgh for 15 years. Mr. Blum had served as a president of the Pittsburgh Section, and was also a member of the Engineers Society of Western Pennsylvania

Edmund Burke Feldman (M. '41) chief engineer for the Hayden-Lee Development Co., of Los Angeles, Calif., died on September 11, at the age of 58. A graduate of the University of Cincinnati, Mr. Feldman taught at the University of Minnesota and, for eight years, was associate professor of civil engineering at Utah State College. He then became connected with John H. Moser of Logan, Utah, and the Public Works Administration at Salt Lake City and San Francisco, and maintained a private practice in Salt Lake City for a brief period. During World War II, Mr. Feldman served overseas as a captain in the U.S. Army.

John Neil Ferguson (M. '08) of Mattapan, Mass., in retirement since 1940, died at St. Petersburg, Fla., on January 6, 1952. He was 79 years old. Mr. Ferguson had been employed by the following New England agencies: The Metropolitan Water Works of Boston; the Charles River Basin Commission of Massachusetts; the Metropolitan Park Commission at Milton; and the Department of Public Works (formerly the Division of Waterways and Public Lands) at Boston, where he was assistant engineer and district waterways engineer for nearly 20 years. Mr. Ferguson was an alumnus of the Massachusetts Institute of Technology.

Harry Homer Harsh (M. '21) employed by the Baltimore & Ohio Railroad Co., continuously since 1907, died in Pittsburgh, Pa., on October 29. He was 67. After graduation from Ohio State University, Mr. Harsh began his association with the railroad, as an assistant in the engineering corps of the Ohio River Division. At the time of his death, he held the position of engineer of maintenance of way, at Pittsburgh. At various times he was division engineer and assistant superintendent, and he had been stationed in Newark and Akron, Ohio, Garrett, Ind., and Baltimore.

Charles Jefferson Hartenstine (A.M. '22) president and chief engineer of the Hartenstine-Zane Co., Inc., of New York, N.Y., died suddenly in his office on June 13. He was 65. Mr. Hartenstine was engaged by the McClintic-Marshall Co., in Pottstown and Pittsburgh, Pa., and the Panama Canal Zone, from 1906 until 1914, and at various other periods during his career. In 1921, he became connected with the Donnell Zane Co., of New York City, and later organized the Hartenstine-Zane Co. He was an alumnus of the University of Pennsylvania and a veteran of World War I.

Harry Hartwell (M. '07) retired engineer of Oakland, Calif., died on October 8, at the age of 83. During his long career, Mr. Hartwell was associated with several firms including Chandler & Co., Inc., of Philadelphia, Pa., and Ford, Bacon & Davis, and Sanderson & Porter, both of New York City. At various times he served as assistant to the president of the Pearson Engineering Corp., Ltd., and as vice-president of K. Taylor Distilling Co., Frankfort, Ky.

Michael Gabriel G. Glen (A.M. '31) since 1936 district materials engineer for the California State Division of Highways at San Bernadino, Calif., died on January 17, 1952, according to information recently received at Society Headquarters. He was 53 years old. Mr. Glen's uninterrupted career in the Division of Highways began in 1928, when he entered it as an instrumentman. He was an alumnus of the University of California.

Joseph Washburn Hawkins (M. '47) engineer of Sebring, Fla., died on December 10, 1951, at the age of 69. Mr. Hawkins had been in engineering practice there since 1944, and was also in Sebring from 1923 to 1930, and in practice in Atlanta, Ga., from 1921 to 1923. At other times he had been with the J. B. McCrary Co., of Atlanta; the Georgia State Highway Department; Alexander Blair, consulting engineer of Lake Placid, Fla.; the U.S. Department of Agriculture; and the Corps of Engineers. He studied at the Georgia School of Technology.

William Henry Hunt (M. '11) retired engineer of New York, N.Y., died on June 20, at the age of 80. Mr. Hunt, who was educated at the Cooper Union, was with several New York firms and the New York Rapid Transit Commission on subway design at the outset of his career. From 1915

(*Continued on page 90*)

1953 SAN FRANCISCO CONVENTION

REGISTRATION FORM

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Name _____ ASCE Member Student Guest
(last name) (please print) (first name)

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* Citizenship (for Tours No. 2 and 3) _____

FUNCTION		NO.	AMOUNT
Tuesday, March 3			
Ladies' Tour & Luncheon—Palo Alto	$3.50		
Wednesday, March 4			
Luncheon, Members and Guests—Fairmont	$3.50		
Convention Banquet, Members and Guests—Fairmont (Limited to 950)	$8.50		
Thursday, March 5			
Ladies' Tour and Luncheon—Nurseries and Country Club	$1.00		
Mens' Tours Choice of one			
Tour No. 1, Power Plant Tour	$2.00		
*Tour No. 2, Aeronautics Tour (limited to 250)	$2.00		
*Tour No. 3, Naval Shipyard Tour (limited to 150)	$1.00		
Friday, March 6			
Ladies' Luncheon and Aquacade—Fairmont	$4.00		
Military Engineers Luncheon—Fairmont	$3.50		
REGISTRATION FEE	$5.00		$5.00
Total			

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Reporting on Unusual Surveying Problems and Their Solutions

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"It can be determined in the field, without calculation, whether or not a survey closes," says John Goggin, C.E. of Marathon, Florida.

On the Keys, John Goggin of Marathon, Florida, uses his Gurley transit for precision work.

"Down here on the Florida keys, land is scarce and credible monuments are few. In the absence of the usual landmarks, accuracy (I prefer the word *precision*) is paramount. We are, therefore, forced to use USC&GS monuments and local plane coordinates.

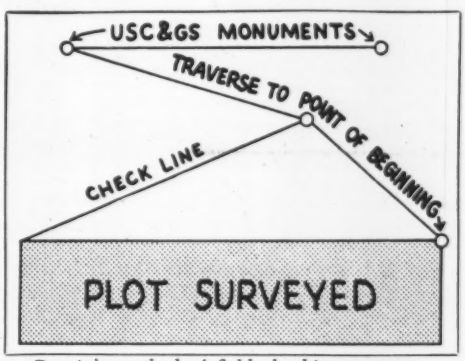
"But ordinary methods with their field calculations are time-consuming. And all field men are not good mathematicians. So we devised a fool-proof method in which the transit does a great deal of the calculating.

"Here's how we do it: Transit is oriented first on a line using the grid azimuth calculated from local point coordinates of USC&GS monuments. Compass is set to read True Bearing of the line. Instrumentman then traverses by azimuth to the point of beginning of the survey. At each change in alignment, a backsight is taken with telescope plunged or inverted—after making certain forward azimuth on the back line is still properly set. The compass is released, telescope erected and sighted on next forward point. The line is measured and azimuth recorded. Compass reading is also recorded as a check.

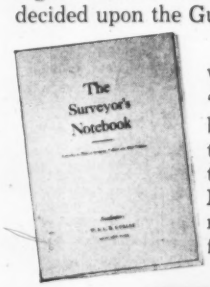
"Now the upper motion is unclamped and a backsight is taken with telescope erect and back azimuth *read*. With the upper motion unclamped, a foresight is again taken. This should check the original azimuth of the forward line.

"This operation is repeated at each change in alignment back to point of beginning, where the forward azimuth should read the same as the calculated value. Thus, *back* azimuths are read each time and *forward* azimuths are double checked. In addition, constant reference to compass readings is made and all observable points-in-survey or known coordinated points are sighted.

"Only an instrument of reputable *precision*, capable of standing hard knocks and rough usage, could be depended upon in performing this work. From long experience, we have decided upon the Gurley."



Goggin's method of field checking a survey.



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Deceased

(*Continued from page 88*)

to 1924 he was sales engineer for the Pittsburgh Bridge & Iron Works, Lehigh Structural Steel, and the Bethlehem Steel Co. In the latter year he became connected with the Frank M. Weaver Co., Inc. of Lansdale, Pa., for whom he served as sales engineer and manager of their New York office until his retirement in 1950.

Robert Smith Johnston (M '24) retired engineer of Yardley, Pa., died on September 26, at the age of 67. Mr. Johnston served as director of research for John A. Roebling's Sons Co., at Trenton, N.J., from 1932 until his retirement in 1946. Previously he had been engineer on research and tests for the Port of New York Authority and engineer physicist for the U.S. Bureau of Standards in Washington, D.C. A graduate of the school of civil engineering at Tufts College, Mr. Johnston had taught at the University of Pennsylvania and Lafayette College for five years.

Clarence Hilton Moore (M. '38) consulting engineer of Winter Haven, Fla., died at his home in that city on June 4, at the age of 57. After graduation from Rensselaer Polytechnic Institute in 1918, Mr. Moore was employed by the Pittsburgh-Des Moines Steel Co., at Des Moines, Iowa, for six years. In 1925, he entered the Aetna Iron & Steel Co., Jacksonville, Fla., advancing to vice-president and chief engineer by 1932 and president by 1936. Mr. Moore was president of the Hoffman Steel Co., Jacksonville, from 1942 to 1948, and president and chief engineer of Hillyer & Covan Inc., also of Jacksonville, in 1949.

Former ASCE Officer, Baxter L. Brown, Dies

Baxter Lamont Brown (M. '03) retired St. Louis engineer and former president of the St. Louis Board of Public Service (1933–1941) died in that city on November 9, at the age of 88. A specialist in railroad engineering and municipal problems, Mr. Brown practiced in St. Louis from 1905 to 1933, and from 1942 to 1948. Prior to 1905, he was connected for almost 20 years with various railroads in the Middle West. Mr. Brown was a Director of ASCE (1921–1923) and vice-president of the American Institute of Consulting Engineers, and had held office in several local engineering societies.

Baxter L. Brown

Richard Bird Ketchum (M. '07) dean emeritus of the School of Mines and Engineering at the University of Utah and an authority on railroad construction and research, died at Urbana, Ill., on November 3. He was 78. A graduate of the University of Illinois, he taught there in 1897 and 1898. His association with the University of Utah began in 1909, when he joined the faculty as instructor. He was professor and head of the department of civil engineering from 1920 to 1927, and dean from 1927 until his retirement in 1939. Dean Ketchum had worked on the construction of the Salt Lake County Memorial Bridge and been with the Salt Lake City Water Works and the Union Pacific and other railroads.

Wilhelm Emil Koch (A.M. '42) construction engineer of New Orleans, La., died on September 9, at the age of 59. Mr. Koch had been connected with Edward F. Neild, Architect, Shreveport, La.; Armstrong & Koch, Architects, and George J. Glover, Inc., both of New Orleans, and James O. Heyworth of Chicago. He was also with several government agencies, including the Mississippi River Commission, the Bureau of Reclamation at Denver, Colo.; and the Bureau of Yards and Docks at Lakehurst, N.J. He was an alumnus of Tulane University, class of 1914.

Arthur Edward La Croix (M. '40) executive vice-president and director of the New England Power Co., and president and director of the Connecticut River Power Co., and the Bellows Falls Hydroelectric Corp., died suddenly on September 5. He was 57. Since his graduation from Cornell University in 1916, Mr. LaCroix had been continuously connected with the New England electric system except for two years with the 301st Engineers in World War I.

Byron James Lambert (M. '19) since 1921 professor and head of the department of civil engineering at the University of Iowa, died on October 29. He was 78. Professor Lambert was a graduate of the Iowa State Teachers College, with a civil engineering degree from the State University of Iowa. Before joining the University of Iowa faculty in 1902 he served as city engineer for Waterloo and Cedar Falls, Iowa. Professor Lambert was the author of several publications on concrete and steel and the inventor of steel stadiums.

Charles Eliot MacKinnon Jr., who had just been accepted as a Junior Member of the Society, but had not yet had time to qualify, was crushed to death by a truck on June 25, while working at the Naval Air Station, Brunswick, Me., as a field engineer for the Thompson & Lichtner Co., Inc., of Brookline, Mass. He was 21 years old and a recent graduate of Norwich University.

Hans William Jorgensen (M. '26) former city engineer of San Diego, Calif., died in that city on November 30, at the age of 77. Mr. Jorgensen, who retired in 1945 after 33 years in the city engineering department, had been city engineer since 1927. Prior to that he was city engineer and mineral surveyor for 10 years in Bisbee, Ariz. Mr. Jorgensen was a past-president of the San Diego Section, and an alumnus of the University of Nebraska.

George Frederick Tongue (M. '43) chief engineer of the Dallas Railway & Terminal Company, died at his home in Dallas, Tex., on October 30. He was 61. Except for a brief period as equipment engineer with the Western Union Telegraph Co., Mr. Tongue had been continuously connected with the D.R.&T. since 1912, serving in the maintenance department for 39 years. In recent years, he had directed all

(*Continued page 92*)

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IN

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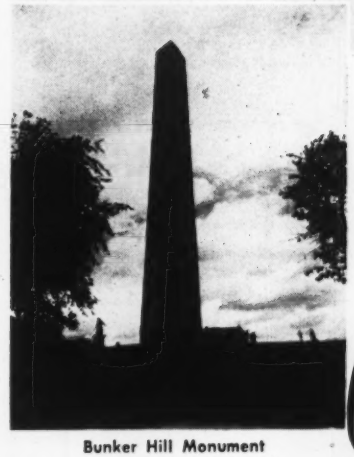
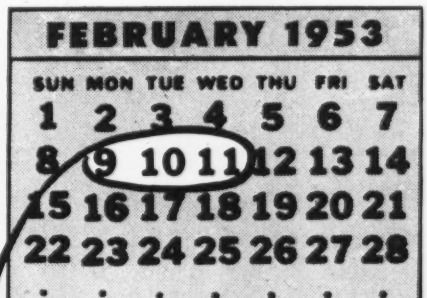
February 9-11, 1953 **HOTEL STATLER**



Bunker Hill Monument



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INTERNATIONAL BUILDING — WASHINGTON 4, D. C.

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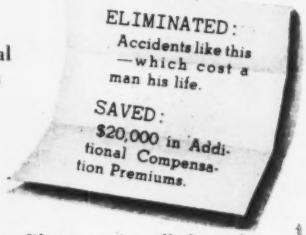
And Ended Slipping Accidents And High Insurance Costs!

It happened at a large industrial plant, where oil and grease on a loading platform created a constant hazard. A lift truck skidded on the slippery surface, toppled from the platform's edge, and crushed the operator to death.



Then A.W. ALGRIP Abrasive Rolled Steel Floor Plate was installed on the platform—and slipping accidents ended at once. For ALGRIP is truly non-slip—even on steep inclines! Hundreds of tiny abrasive particles in each square foot of ALGRIP converted the slippery, dangerous platform into a hard gripping, anti-skid surface—safe for men and vehicles alike.

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Deceased

(*Continued from page 90*)

design, construction, and maintenance work and since 1951 had been chief engineer. Mr. Tongue was a graduate of Oklahoma A & M College and a veteran of World War I.

Benjamin Cleveland Leadbetter (M. '42) engineer consultant with Graham Bros., Inc., of El Monte, Calif., died recently at the age of 72. During his early career, Mr. Leadbetter worked in mines located in Nevada, the Transvaal of South Africa, and Chile. He had also been employed by the Metropolitan Water District of Southern California for several years. Mr. Leadbetter received the degree of bachelor of science from the University of Nevada.

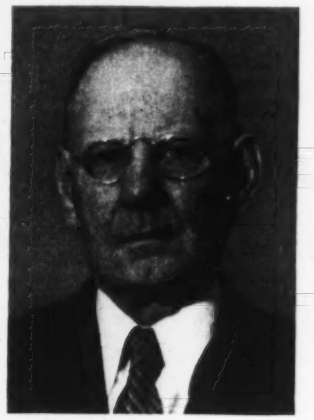
James Augustus Nelson (M. '09) retired engineer of Mount Dora, Fla., died on April 14, at the age of 84. Following graduation from the Sheffield Scientific School at Yale University in 1888, he worked for several organizations including the Hotchkiss Ordnance Co., the U.S. Navy Department, and the Wm. R. Trigg Co., of Richmond, Va. From 1910 to 1917 and from 1920 to 1922 he was vice-president of the East Jersey Pipe Co., of New York and Paterson, N.J.; from 1922 to 1935, with the McClintic-Marshall Corp.; and from 1935 to his retirement in 1940, with the Bethlehem Steel Co.

Frederic Emery Pierce (M. '09) retired engineer of New York, N.Y., died on September 8, at the age of 82. For more than 30 years, Mr. Pierce had been in private practice as a consulting engineer in New York. Earlier in his career he was employed by the New Jersey Zinc Co., for several years, advancing to engineer of construction. Mr. Pierce was an alumnus of the Columbia University School of Mines.

Walter Ferrell Winton (Aff. '18) colonel, U.S. Army (retired), died at his home in Columbus, N.Mex., on May 12, at the age of 65. Colonel Winton who retired from the Army in 1945 after 34 years of service, saw active duty with the Pershing expedition in Mexico, in World War I, and for several years was military attaché to Peru and Bolivia. Earlier he had worked on the National Railway at Durango, Mexico. Colonel Winton was a graduate of Vanderbilt University and also held a master's degree from the University of Toulouse.

Jacob Latch Warner (M. '17) engineering consultant of Wilmington, Del., since his retirement in 1946 from E. I. du Pont de Nemours & Co., died in Wilmington on November 11. He was 76. In 1904, following first employment with the New York Shipbuilding Co., New York, N.Y., Mr. Warner began a 42-year association with E. I. du Pont de Nemours & Co. in the engineering department. He was transferred to the real estate department as manager in 1930, and at the time of his retirement was special assistant in the service department. Mr. Warner was officer in several business enterprises, including the American Superior Products, Inc., the Delaware Plastics Co., Mid-City Building Corp., and the Community Realty Co. He was a graduate of the University of Pennsylvania.

NEWS OF ENGINEERS



W. H. McAlpine retires as special assistant to Chief of Engineers in Washington, D. C., after 50 years in Army Corps of Engineers. Mr. McAlpine, an Honorary Member of ASCE, is an international authority on construction of dams, reservoirs and levees. During his career he has been district engineer at Louisville, Ky., and St. Louis, and has been special assistant in Washington since 1934.

Paul D. Berrigan, colonel in the Corps of Engineers, stationed in the Office of the Joint Chiefs of Staff, Washington, D.C., since 1951, has been assigned as South Pacific Division Engineer, with headquarters in San Francisco, Calif.

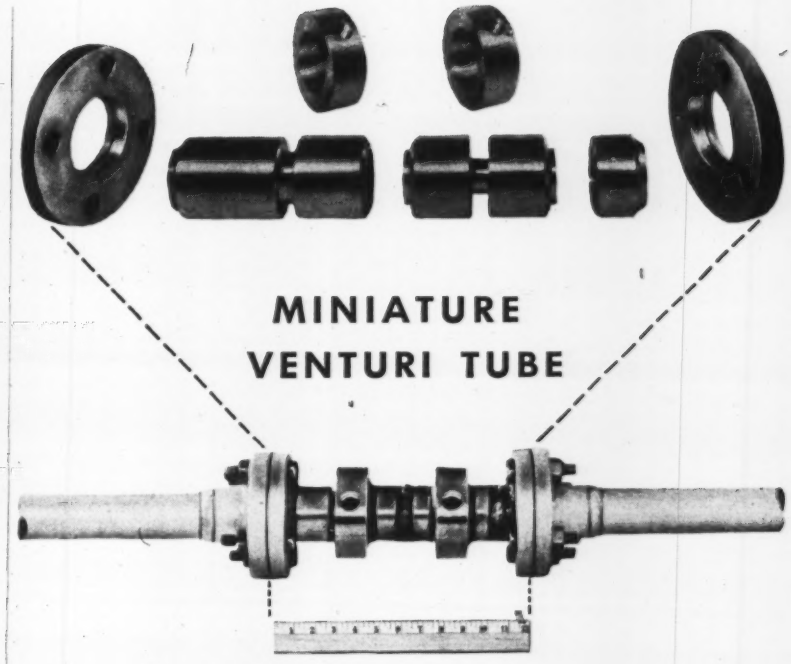
Joseph D. Blatt has been promoted from chief of the planning staff division of the Civil Aeronautics Administration Office of Federal Airways, to assistant administrator for program coordination. He has been active in coordinating aviation policies in the U.S. Government's Air Coordinating Committee, and in the International Civil Aviation Organization at Montreal, where he is currently heading the U.S. delegation to a session of the Aerodrome, Air Route and Ground Aids Division.

William J. Carroll, borough engineer for Hollidaysburg, Pa., for the past several years, has resigned to accept a job with an engineering firm in Philadelphia.

T. C. Forrest, Jr., member of the consulting firm, Forrest & Cotton of Dallas, Tex., and immediate past-president of the Texas Section of ASCE, was recently nominated president of the National Society of Professional Engineers for 1953.

Howard S. Gay is retiring as chief of engineering planning for the North Atlantic Division of the Army Corps of Engineers, with headquarters in New York City, after a

(*Continued on page 94*)



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News of Engineers

(*Continued from page 93*)

long career in the Corps of Engineers. Mr. Gay has directed the planning of such projects as Austin Dam in Texas and the electrification of the Island of St. Croix in the Virgin Islands, and was instrumental in the planning of the proposed St. Lawrence Seaway project.

Richard E. Dougherty, Past-President of ASCE and retired vice-president of the New York Central Railroad, was the recipient of the 1952 Lion Award given by the Columbia University Alumni Club of Central Westchester. Mr. Dougherty, a member of the class of 1901 of the Columbia University School of Engineering, will be cited "for outstanding service to his community, his profession and his university."

Harold D. Hauf recently completed a tour of active duty with the Civil Engineer Corps of the Navy and was released from his duties as assistant district public works officer and deputy officer-in-charge-of construction, First Naval District, Boston, Mass. He has returned to Yale University to become managing engineer of a classified and urgent research project conducted for the Office of Naval Research.

Grant M. Hinkamp, formerly engineer for Lawrence Peterson & Associates, Milwaukee, Wis., is now with Floyd G. Browne and Associates, Marion, Ohio, in the capacity of chief construction engineer.

Charles H. Hunnell, of Morristown, Ind., has been recalled to active duty with the Corps of Engineers, with the rank of lieutenant colonel. Colonel Hunnell has been with the State Highway Commission of Indiana for over 25 years—recently as assistant engineer of road construction.

J. A. Bradley has resigned as flood-control engineer for Orange County (Calif.) because of ill health. After a few months' rest, he will open an office in Santa Ana, Calif., where he will specialize in hydraulics and consult in management problems of special districts. Mr. Bradley has been on the staff of the Orange County Flood Control District since 1929, and has been head of the department since 1946.

William O. Hiltabidle, Jr., vice admiral, U.S. Navy, and district civil engineer with headquarters at San Diego, Calif., has retired after many years of service, and has accepted the position of assistant to the president of the contracting firm of Charles H. Tompkins Co., Washington, D.C.

Lowell J. Stephenson, construction engineer for the consulting firm of Porter-Urquhart Associated, has been appointed project engineer for the organization's contract with the New Jersey Highway Authority on construction of Section 8 of the Garden State Parkway. Mr. Stephenson recently completed a three-year assignment as resident engineer for Edwards & Kelcey, Frederic R. Harris, Inc., and O. J. Porter & Co., on the construction of Section 6 of the New Jersey Turnpike for the New Jersey Turnpike Authority.

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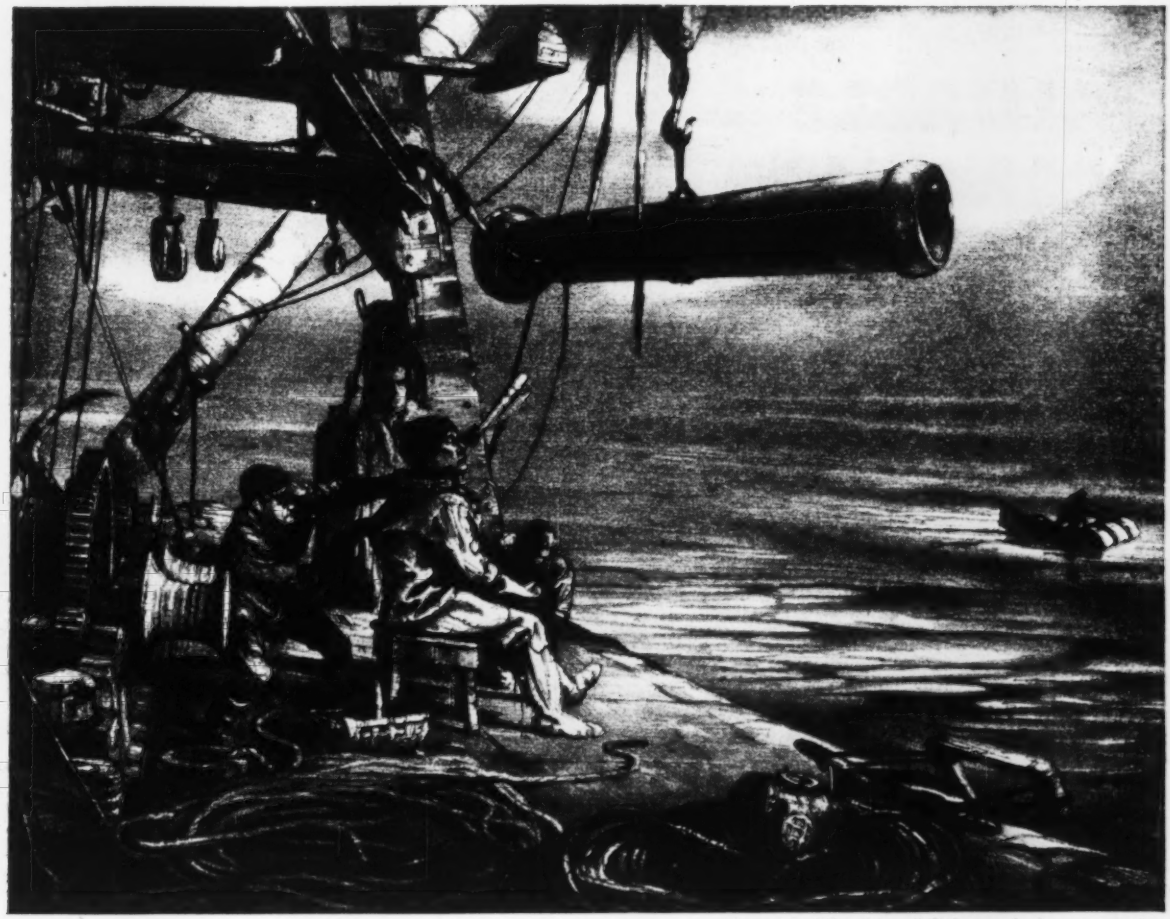

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VALUABLE MANUAL

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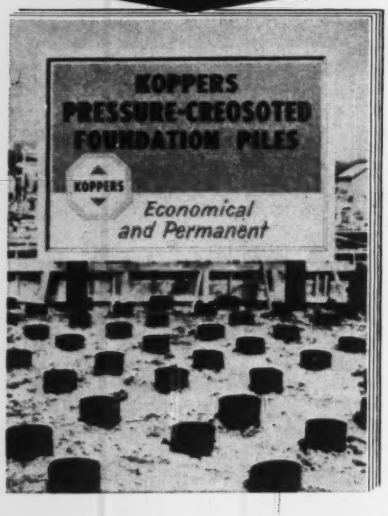

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News of Engineers

(*Continued from page 94*)

Omar J. Loeltz, who has been acting district engineer for the Ground Water Branch, U.S. Geological Survey, at Carson City, Nev., since July 1952, has been designated district engineer. He joined the Ground Water Branch at Roswell, N. Mex., in 1939 and was on active duty with the Navy during World War II. Upon his return to the Survey in 1946, he was assigned to the Nevada district, as assistant to the district engineer.

Harold R. Orr, office engineer for the U.S. Bureau of Reclamation on the construction of Davis Dam in Nevada, has been designated acting construction engineer on the project.

Palmer W. Roberts, commander, CEC, USN, on duty in the Bureau of Yards and Docks, Washington, D.C., for the past two years, has been assigned as assistant director of the Pacific-Alaskan Division of the Bureau. Commander Roberts, a resident of Chicago, will coordinate Naval shore establishment construction in the Pacific-Alaskan area.

D. B. Steinman, New York City consultant, was elected president of the New York Academy of Sciences at the annual dinner of the Academy.

Jack D. White is now employed as structural engineer by the Stearns-Roger Manufacturing Co., Denver, Colo. He recently resigned his position as structural engineer, Branch of Design and Construction, U.S. Bureau of Reclamation, at Denver after serving with the Bureau almost seven years.

Edward C. Cahaly, former engineer with McKoy Helgerson Co., in Greenville, S.C., is now working as a civil engineer for J. E. Sirrine and Co., on the Bowater paper mill job at Charleston, Tenn.

Charles R. Richardson, previously with the City of Oklahoma City, Okla., as an engineer and draftsman associated with the Floodway project, is now with the Atlas Engineering Corp., at Oak Ridge, Tenn.

Paul R. Speer has been transferred from the St. Paul, Minn., office of the U.S. Geological Survey, where he was district engineer, to their office in Chattanooga, Tenn.

Omer L. Deweese, project manager for the Maxon Construction Co., Inc., at Franklin, Ohio, is now chief engineer on general and heavy construction for the company at Oak Ridge, Tenn.

James Spofford, since 1949 superintendent of the Minidoka project in Idaho for the Bureau of Reclamation, is retiring. Mr. Spofford has been connected with the Bureau in Idaho and eastern Oregon for eleven years, serving as irrigation manager of the 100,000-acre Owyhee Project and as state reclamation engineer for Idaho.

James W. Pastorius has been elected vice-president and plant manager of the Whitehall Cement Manufacturing Co., at Cementon, Pa. Connected with the company since 1936, Mr. Pastorius has been plant manager for the past year and a half.

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RECENT BOOKS

Changes in Characteristics of Portland Cement as Exhibited by Laboratory Tests Over the Period 1904 to 1950

This paper by H. F. Gonnerman and William Larch constitutes ASTM Special Technical Publication No. 127. It describes the changes in composition, fineness, and strength-producing characteristics of Type I portland cement. Most of these changes are shown to have occurred during the period, 1926 to 1940. Included are data comparing composition, fineness, strength-producing characteristics, volume change, heat of hydration, and sulfate resistance of the five types of portland cement covered by ASTM Specifications C 150–49. (American Society for Testing Materials, 1916 Race Street, Philadelphia 3, Pa., 56 pp., $1.)

C R S I Design Handbook

The object of the C R S I Design Handbook by R. C. Reese is to present finished designs of reinforced concrete members, giving concrete sizes and reinforcement. The extensive presentation of tables, diagrams, and other technical data is divided as follows: Reinforcing bars, design formulas, design methods, specialized design data, ASTM specifications, and over 250 pages of detailed safe-load tables covering slabs, joists, beams, columns, footings, and retaining walls. The designs given are particularly good for preliminary estimating, for establishing sizes and clearances, and for comparing different types of construction. (Concrete Reinforcing Steel Institute, 38 South Dearborn St., Chicago 3, Ill., 1952. 412 pp., $5.)

Stahlbau-Handbuch 1952

The *Steel Construction Handbook* (formerly called *Steel Construction Calendar*) provides a wide variety of reference data under the following groupings: Fundamentals—mathematics, etc.; statics; current German standards and regulations concerned with steel construction, covering buildings, crane structures, railroad and highway bridges, etc.; profile tables; connections and rivet tables. A new, comprehensive German standard on "design fundamentals for stability" is appended. Edited by G. Unold and F. Kleineberg, sponsored by Deutscher Stahlbau-Verband. (Industrie-und Handelsverlag Walter Dorn, Bremen-Horn, Germany, 1952. 657 pp., DM 28.00.)

Traite D'Irrigation

This comprehensive treatise by V. Bauzil begins with the study of requirements and conditions imposed by the crops and physical areas to be irrigated. It reviews the hydraulic formulas involved, the layout of irrigation systems, canal construction, dams and other auxiliary structures, drainage problems, and the utilization of underground water for irrigation. (Editions Eyrolles, Paris, 1952. 2 vol. 412 pp., frs. 5500.)

Verdichten von Leichtbeton Durch Rutteln

Deutscher Ausschuss für Stahlbeton. Heft 108.

In this contribution Kurt Walz presents the results of laboratory tests on the compaction of lightweight concrete by vibration. It discusses the effect of vibration-compaction on compressive strength, gross specific weight, and stability (creep strength) of lightweight concrete in relation to vibration rate, amplitude, and length of time, and also in relation to concrete stiffness, composition, and aggregates. (Wilhelm Ernst & Sohn, Berlin, 1952. 27 pp., DM 8.00.)

Mechanics of Fluids

In the second edition of this introductory textbook by Glenn Murphy, on the behavior of fluids, the approach and techniques are

(*Continued on page 100*)

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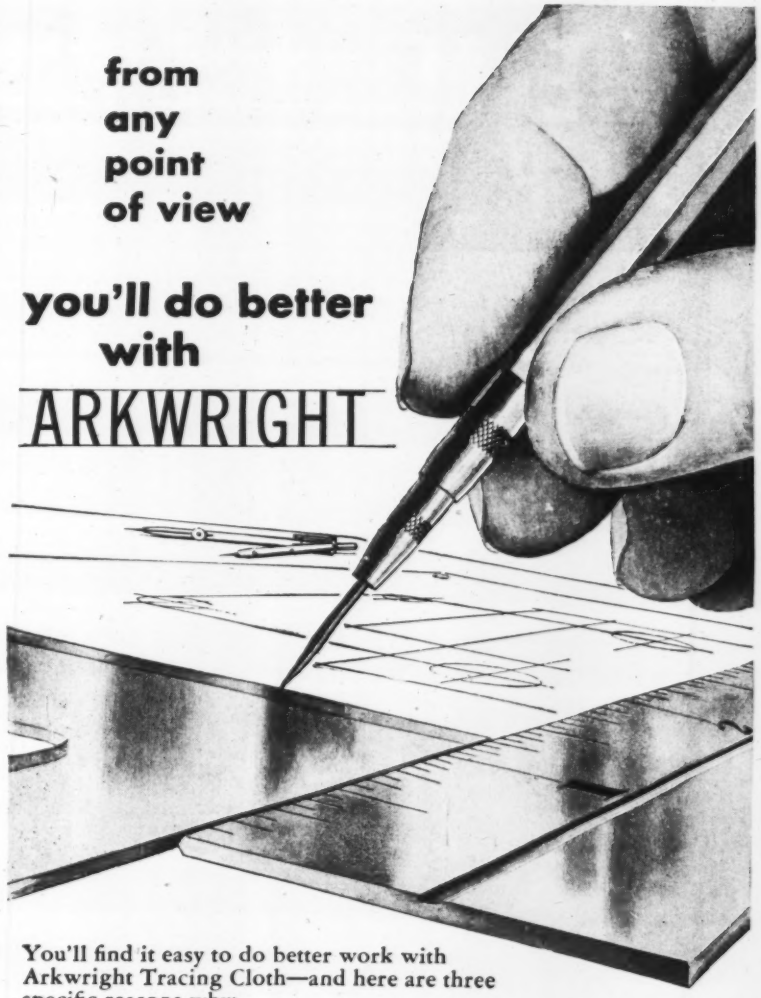

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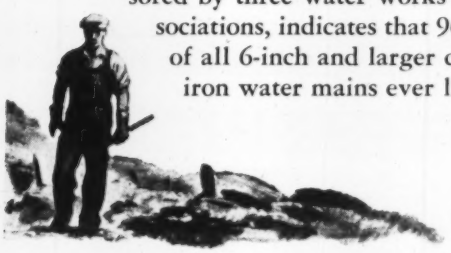

in 25 representative cities are still in service. And when answers to a questionnaire, mailed to gas officials in 43 large cities, show that *original* cast iron mains are still in service in 29 of the cities.

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SERVES FOR CENTURIES

Recent Books

(*Continued from page 97*)

those which have proved successful in the mechanics of solids. The basic method of analysis is that of the free-body, used in conjunction with the fundamental principles of mechanics, expressed in Newton's laws of motion. Numerous practical applications of the theory are cited, and numerical and laboratory problems are provided. Important revisions are in the material on viscosity, flow-nets, jets, fluid couplings and automatic transmissions. (International Textbook Company, Scranton 9, Pa., 1952. 309 pp., $6.)

Reinforced Concrete

In *Reinforced Concrete*, Oscar Faber surveys the materials, properties, and uses of reinforced concrete and describes current methods of design, construction, and application. Included are fundamental theory, specific design data, and numerous practical observations based on the author's experience. Separate chapters are devoted to special topics such as shell concrete, chimneys, prestressed concrete, silos, bunkers, etc. Tables and graphs are extensively used. (E. & F. N. Spon, Ltd., 22 Henrietta St., London, W.C. 2, England, 1952. 232 + 55 pp., 30 s.)

Water

A study of the properties and constitution of water, its circulation on the earth, and its utilization by man. From salinity to sedimentation, from meteorology to water control by dams, the author, Cyril S. Fox, covers a wide range of topics, presenting a remarkable amount of statistical and analytical data in condensed form. The book is intended as the introductory volume of a series on the practical aspects of water supply. (Philosophical Library, 15 East 40th St., New York, 16, N. Y., 1952. 148 pp., $8.75.)

World Population and Future Resources

The proceedings of the Second Centennial Academic Conference of Northwestern University, edited by Paul K. Hatt, comprise this volume of 20 papers broadly classified as follows; The population factor, food resources, material resources of industry, and energy resources. In addition to a logical presentation of the problem posed by our expanding world population, certain technological aspects of the problem are emphasized in separate papers—mineral resources and exploitation, the future of structural materials, new products, liquid fuels for the future, solar and economic energy in the world economy, and food technology. (The American Book Co., 88 Lexington Ave., New York, N. Y., 1951, 262 pp., $3.50.)

ASTM Specifications for Steel Piping Materials

This compilation, prepared by ASTM Committee A-1 on Steel, contains all ASTM specifications for carbon-steel and alloy-steel pipe and tubing. It covers pipe to convey liquids, vapors, and gases at normal and elevated temperatures, still tubes for refinery service, and tubes for boilers, heat exchangers, condensers, and superheaters. Specifications are also included for castings, forgings, bolts, and nuts used in piping installations. A number of emergency alternate provisions in separate form are provided with the book or will be sent subsequently. (American Society for Testing Materials, 1916 Race St., Philadelphia 3, Pa., 1952. 372 pp., $3.50.)

Engineers and Ivory Towers

In *Engineers and Ivory Towers*, Robert C. Goodpasture has made a compilation in essay form of the non-technical writings and lectures of an eminent teacher and structural engineer, Hardy Cross. The content reflects the author's opinions on the relationship of engineer and engineering to science and the humanities, touching on a wide range of subjects including education, graduate study, standardization, and the responsibilities and obligations of engineers. (The McGraw-Hill Book Co., Inc., 330 West 42nd Street, New York 36, N. Y., 141 pp., $3.50.)

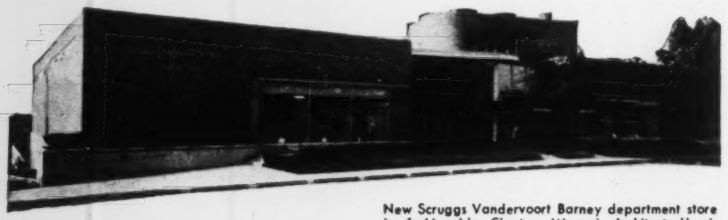
Piping Design and Engineering

The plan of this book has been to compile in a single publication engineering data and technical information for the use of engineers engaged in the design and application of pressure-piping systems hitherto available only by consulting a number of sources. Major sections are as follows: Expansion and stresses; velocity and pressure drop; pressure-temperature pipe graphs; piping materials; pipe fabrication; pipe hangers and supports; and general tables. Considerable material not previously published has been included. (Grinnell Company, Inc., Providence, R. I., 1951. 221 pp., $10.)

Pratische Probleme der Baustatik

Practical problems in the field of structural statics with solutions by the method of influence lines are presented by Othmar Weiss in this volume. Part I covers structures with stable or vertically displaceable panel points and Part II frame structures with displaceable panel points. A tabular appendix gives moments of inertia of reinforced-concrete T-beams and supplementary tables for the calculation of frame structures and continuous beams. (Franz Deuticke, Vienna, Austria, 1952. 193 pp., DM 6.00.)

(*Continued on page 102*)



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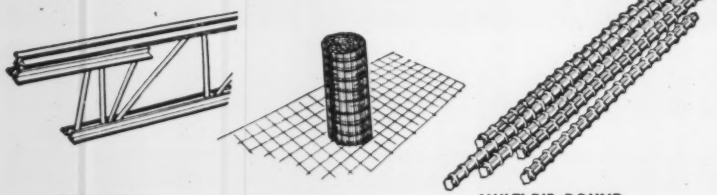

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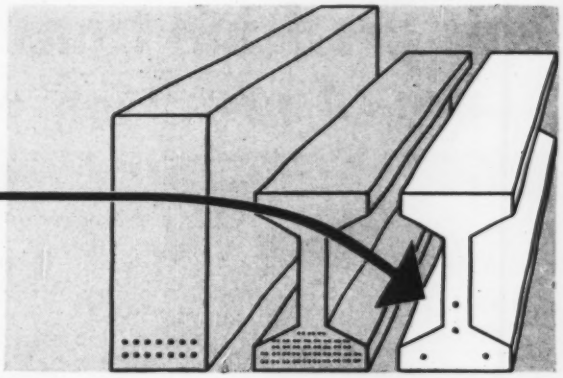

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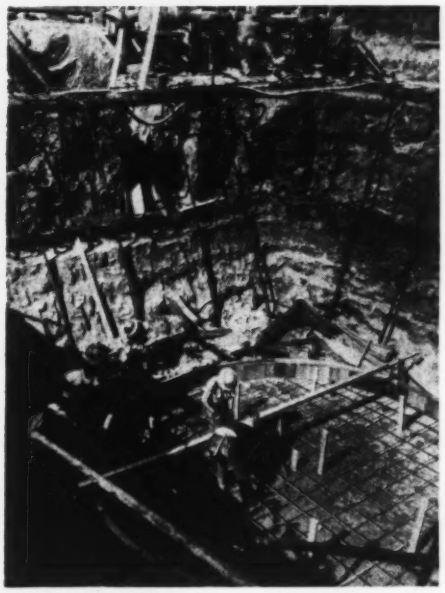

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Recent Books

(*Continued from page 100*)

(A) Bibliographical Survey of Flow Through Orifices and Parallel-Throated Nozzles

More than 600 entries classified under sixteen subject headings comprise this bibliography, B.S.I.R.A. Research Publication, M.9, by T. H. Redding, which covers the period 1832–1946. The references are arranged in chronological order under each heading and are annotated for more effective use, and cross references are supplied to allied material under other headings. There are also an explanatory technical introduction, an author index to the bibliography, and a glossary of terms used in flow-metering practice. (Chapman & Hall Ltd., 37 Essex St., W.C.2, 1952. 196 pp., 32. 6d.)

Atomic Power

Beginning with a brief résumé of the technical background, the authors continue with a factual analysis of the economic, sociological, political, and geographical aspects of atomic power development. Part II deals with costs and industrial applications, utilizing the iron and steel industry and the aluminum industry as case studies. Part III discusses the relation of atomic power to the process of economic development and its impact upon regional economies, with Brazil particularly considered as a case study. Authors are Walter Isard and Vincent Whitney. (The Blakiston Co., 575 Madison Ave., New York 22, N. Y., 1952. 235 pp., $4.75.)

Civil Engineering Plant and Methods

Equipment and methods for basic civil engineering operations are described in this volume, by Rolt Hammond, and illustrated under the following classifications: Excavating plant, piling and foundations, concrete mixing and placing, cranes and other lifting appliances, dock and harbor construction, tunneling, unit construction work, welding, and roadmaking. There is also a special chapter dealing with the organization of civil engineering work. (Ernest Benn Ltd., London, 1952. 229 pp., 25s.)

Symposium on Surface and Subsurface Reconnaissance

A group of fifteen papers dealing with geological, pedological, airphoto, or geophysical interpretations of data on earth formations and deposits, over half of the material being devoted to geophysical methods. The object of the symposium was to establish a groundwork for defining the applications and limitations of these major approaches to reconnaissance with respect to the accuracy of their results. The panel discussions on the papers are included. (American Society for Testing Materials, 1916 Race Street, Philadelphia 3, Pa. S.T.P. No. 122, 1952. 228 pp., $3.)

Theory of Elasticity and Plasticity. (Harvard Monographs in Applied Science, No. 3.)

Suitable for first-year graduate students, this book by H. M. Westergaard is written from the point of view of the engineer although the treatment is entirely theoretical. Topics covered include the important basic concepts of the states of stress and strain, the basic laws and equations of elasticity and plasticity, and specialized applications to hollow cylinders, spheres, etc., to various force effects, and to thermal stresses. (John Wiley & Sons, Inc., 440 Fourth Avenue, New York 16, N.Y., 1952. 176 pp., $5.)

Zauberwelt der Normzahlen

The author, Wilhelm Strahringer, describes a variety of uses for "preferred number" series in the fields of production and distribution, economics and management, and any other activities to which standardization of this sort may be applied. Graphs are freely used for efficient representation of these applications. The official German standard series of preferred numbers, of which the base term is 1 and the ratio is $\sqrt[40]{10}$, is the particular series on which the book is based. (Verlags-und Wirtschaftsgesellschaft der Elektrizitätswerke, Frankfurt-am-Main, Germany, 1952. 95 pp., tables, charts, DM. 6.50.)

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CITY ENGINEER-DIRECTOR PUBLIC WORKS; M. ASCE; B. S., sanitary engineering, University of California; licensed in California; 25 years' experience in municipal engineering, administration, supervision and design of sewers, streets, refuse disposal, and the maintenance thereof. C-802-5211-A-4-San Francisco.

CIVIL AND STRUCTURAL ENGINEER; A. M. ASCE; 33; B. S., Mississippi State College, 1941; 11 years' experience structural design, field engineering, construction, contract administration; present position, Contract Officer of Bureau of Yards and Docks Contracts, USN; to be released in January 1953; registered in Florida and Louisiana. Desires employment as structural and/or civil engineer with engineering or industrial firm in Southeast. C-803.

Positions Available

INSTRUCTOR IN CIVIL ENGINEERING, to teach in the fields of fluid mechanics and soil mechanics. Should have at least an M. S. degree, preferably a doctor's degree. Must be a citizen. Salary open. Location, Connecticut. Y-7274.

ASSISTANT OR ASSOCIATE PROFESSORS, with master's degree in civil engineering, Ph. D. desirable, to teach soil mechanics, structures and general civil engineering. Some teaching experience desirable; also some experience in industry. Salary, $6,000–$7,500 for 9 months. Location, Delaware. Y-7369 (*a*).

RESEARCH ASSISTANT, recent civil engineering graduate, for work on research project involving the investigation of structural members for use in supporting structures for ground electronic operations. Duties include preparation of engineering drawings, engineering computations, conducting lab tests and assembling resulting data. Opportunity to pursue graduate work toward master's degree in C. E., and also to instruct in laboratory work during the school year. Salary open. Location, New York Area. Y-7386.

ENGINEER, construction, structural or civil, under 45, technical graduate, with 2 years' experience in construction engineering work for field and office work in connection with building construction. Salary, $3,600–$5,400 a year. Location, New York, N.Y. Y-7600 (*a*).

CONSTRUCTION SUPERINTENDENT, not over 47, M. E. or C. E. degree, with at least 10 years' experience in heavy construction involving a considerable length of time in full charge of the projects. Must have had petroleum refinery or heavy industrial chemical plant experience. Will do 100 percent field work at the actual construction site. Salary, $7,200–$7,500, plus expenses. Locations, throughout the United States. Y-7626.

SANITARY ENGINEER, 30–45, graduate, with degree in sanitary engineering or civil engineering with minor in sanitary engineering, or chemist, and at least 5 years' experience in a waterworks system, public health activities in sanitary engineering sales. Will act as assistant to the water supply superintendent to supervise the water purification and sanitary engineering sales. Must be U. S. citizen. Salary, $5,388–$6,644 a year to start. Location, Florida. Y-7664.

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SANITARY ENGINEER with experience to design and lay out sewage disposal plants and systems. Salary, $5,000–$6,000 a year. Location, New York, N.Y. Y-7839.

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ENGINEERS. (*a*) Construction inspector, graduate, with degree in civil engineering. General knowledge of industrial construction methods and practices and building codes, preferably including work in petroleum refinery. Thorough familiarity with engineering standards and specifications and inspection methods. Experience in hiring and supervising men on construction projects. Salary, $8,300 a year. Location, New York State. (*b*) Construction inspector (regional), 25–35, B. S. degree, preferably in mechanical or civil engineering, to review plans, specifications, and contracts for new construction and additions or alterations to existing service stations, bulk plants and terminals in marketing region. Contacts with contractors relative to receiving, inspecting and procuring materials, etc. Salary, $5,044 a year. Location, Pennsylvania and New York. Y-7960.

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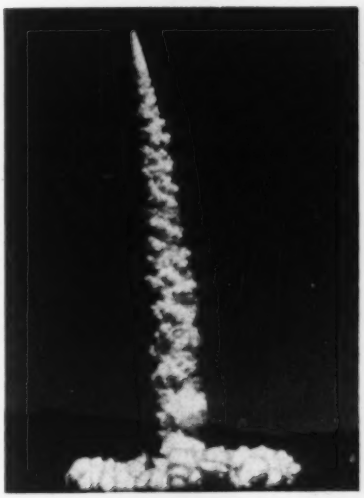

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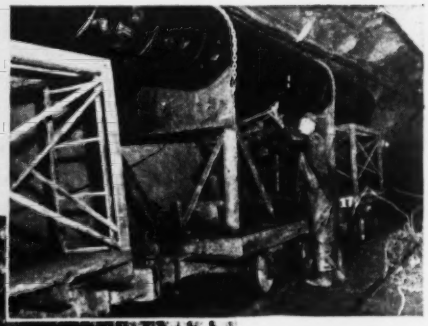

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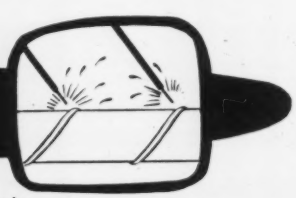

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Non-ASCE Meetings

American Road Builders Association. Headquarters for the 1953 annual meeting of the American Road Builders Association, to be held February 9–11, will be the Hotel Statler, Boston, Mass.

American Society of Heating and Ventilating Engineers. The 11th International Heating and Ventilating Exposition, which is scheduled for the International Amphitheatre, Chicago, Ill., January 26–30, is being sponsored by the American Society of Heating and Ventilating Engineers, during the period of the society's 59th annual meeting. Headquarters for the meeting will be the Conrad Hilton Hotel.

American Institute of Electrical Engineers. The American Institute of Electrical Engineers will hold its winter general meeting at the Hotel Statler in New York, N.Y., January 19–23.

Society of Automotive Engineers. The annual meeting and engineering display of the Society of Automotive Engineers will take place at the Sheraton-Cadillac in Detroit, Mich., January 12–16.

Second Annual Georgia Highway Conference. The second annual Georgia highway conference—jointly sponsored by the Georgia Highway Department and the School of Civil Engineering, Georgia Institute of Technology—will take place on the college campus, January 26–28. Further information may be obtained by writing to C. H. Taylor, Engineering Extension Division, Georgia Institute of Technology, Atlanta, Ga.

Society of Plastics Engineers, Inc. The ninth annual technical conference of the Society of Plastics Engineers, Inc., will be held at the Hotel Statler in Boston, Mass., January 21–23.

Western Computer Conference. The first meeting of the Western Computer Conference will take place at the Hotel Statler in Los Angeles, Calif., February 4–6. This conference will be sponsored by the joint Computer Conference Committee of the Institute of Radio Engineers and the American Institute of Electrical Engineers.

American Concrete Institute. The Statler Hotel in Boston, Mass., will be headquarters for the 49th annual convention of the American Concrete Institute, from February 17–19.

Chi Epsilon. A meeting of the New York Alumni Chapter will be held in the Engineering Societies Building, Room 1101, 33 West 39th St., New York, N.Y., on February 4, at 7:30 p.m. It will be preceded by an informal dinner in the New York Times Dining Room, 11th floor, 229 West 43rd St., at 6 p.m.

American Society of Landscape Architects. Annual meeting of the American Society of Landscape Architects will be held at the Ansley Hotel, Atlanta, Ga., from January 25–28.

Positions Announced

Arizona State Department of Health. Announcement is made by the Arizona State Department of Health of the availability of a position as sanitary engineer at a salary ranging from $390–465 a month. A degree in engineering is required and some experience in the field preferred. Inquiries should be addressed to the Merit System Council, 1632 West Adams, Phoenix, Ariz.

Applications for Admission to ASCE, Nov. 22–Dec. 13

Applying for Member

ROBERT HENRY ALBRECHT, Wilmington, Del.
RAY NASON BOOKS, Flagstaff, Ariz.
DENNIS WILLIAM BROSNAN, Washington, D.C.
JOHN JOSEPH CASHMAN, JR., Washington, D.C.
WILLIAM ELSWORTH COLLINS, Denver, Colo.
GEORGE WARREN CUMBUS, Greenville, S.C.
FRANKLIN LEROY DAVIS, Juneau, Alaska.
WILLIAM DAVIS, New York, N.Y.
GEORGE EDSON DUTTON, Shreveport, La.
CLIFFORD WAYNE ESHBAUGH, Rolla, Mo.
HENRY GRACE, London, England.
ROBERT NORRIS GRUNOW, Atlanta, Ga.
JAMES ROBERSON HORTENSTINE, Juneau, Alaska.
WATSON WAH SUNG LEE, Honolulu, T.H.
JACOB HENRY LEON, New York, N.Y.
WILLIAM STRATHERN MACKINTOSH, St. Paul, Minn.
HENRY LANE MITCHELL, Shreveport, La.
THEODORE LYNN MOORE, New York, N.Y.
DONALD KING MORTON, Corning, N.Y.
TINO JOSEPH POGGIANI, Indianapolis, Ind.
JAMES NICHOLAS SAVAGE, Detroit, Mich.
ERIC SCHAEFER, San Francisco, Calif.
ROLF SCHJODT, Rio de Janeiro, Brazil.
U KYAN SEIN, Rangoon, Burma.
HENRY SEITZ, Baltimore, Md.
CLYDE TIMOTHY SULLIVAN, Washington, D.C.

Applying for Associate Member

JAMES ROBERT ADAMS, Dhahran, Saudi Arabia.
ELROY CHARLES BALKE, Kansas City, Mo.
HEINZ LUDWIG BENZEL, Wyandotte, Mich.
HOWARD WILLIAM BROWN, Freeport, Tex.
HENRY ELMER FUEHRER, Charleston, W. Va.
WARREN KEITH GERHART, Balboa, C.Z.
JIMMY KOSTOFF, Logan, Utah.
KAI-CHENG LU, Norwalk, Calif.
KENNETH MARKWELL, JR., Memphis, Tenn.
JEAN JACQUES MARTIN, Paris, France.
KOKA JAGANNADHA RAO, West Godavari Dist., South India.
CARLOS RANALDI REPETTO, Columbus, Ohio.
WILLIAM WEST SHULL, Freeport, Tex.
JOHN WILLIAM SMITH, Powell, Tenn.
WILLIAM LATON STEPHENS, Boise, Idaho.
ALBERT THEODORE TOMS, Billings, Mont.
HARVEY ALFRED TOWNE, Sacramento, Calif.
PAUL ROBERT WATSON, JR., Sacramento, Calif.
JOHN MILTON WEBER, St. Louis, Mo.
CHARLES WILLIAM YAEGER, Big Bear Lake, Calif.

Applying for Junior Member

JOHN CARDIFF ARCHER, Fort Belvoir, Va.
RAYMOND LUTHER BAKER, Charleston, S.C.
WILLIAM DOUGLAS BELL, Fort Belvoir, Va.
JOE ALBERN CARTER, JR., San Antonio, Tex.
ROBERT JAMES CROWTHER, Houston, Tex.
CHARLES FRANKLIN KAY, Syracuse, N.Y.
ROBERT RAYMOND KUSKE, Richmond, Va.
ROSCOE NIGEL MILLER, JR., Ocala, Fla.
ROBERT KAZUO NAGATO, Honolulu, T.H.
ALBERT JOHN OMAN, Duluth, Minn.
WILLIAM ALLEN RODGER, Portland, Oreg.
ROBERT EARL SELLECK, Cleveland, Ohio.
METE AVNI SOZEN, Oakland, Calif.
ADRIANUS VAN KAMPEN, Lansing, Mich.
WILLIAM VAZQUEZ AGRAIT, Mayaquez, Puerto Rico.
KENNETH WILLIAM WHITNEY, Lancaster, Pa.

[Applications for Junior Membership from ASCE Student Chapters are not listed.]

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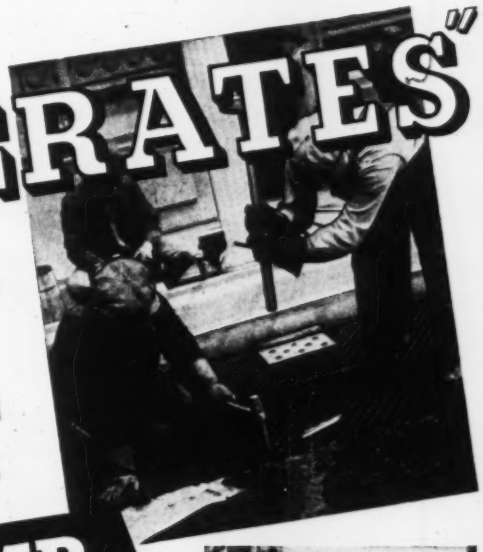

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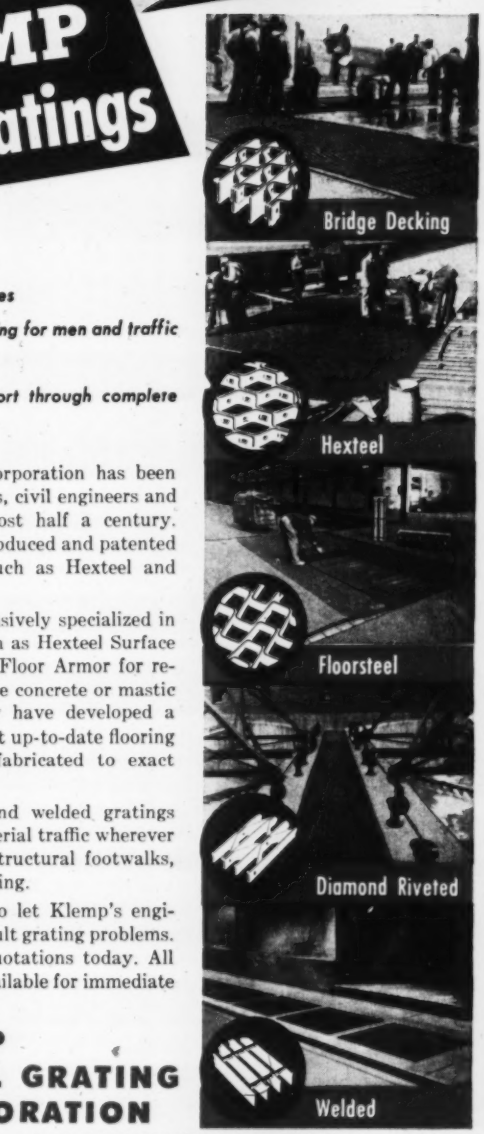

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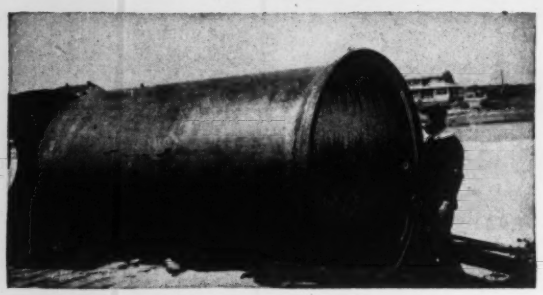
Short-span precast and prestressed beams carry highway bridge . . .

This article begins on page 37

(*Continued from page 41*)

contract, including four experimental bridges on the Newburyport Turnpike, is that, like the automobile, prestressing is here to stay. The tests have borne out the design criteria except that the loss in prestressing on Danvers indicates an allowance of 15 percent may be insufficient.

This is worth looking into



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The most significant conclusion from the tests is indicated by the load deflection curves, Fig. 5. The behavior of the three beams up to cracking, or about twice the live load, was remarkably consistent. The two unbonded beams followed each other closely to the ultimate, but from the cracking load on, the beam in which cables were grouted was much stiffer, and the overload capacity at ultimate load was about 18.5 percent more.

There is a difference of opinion at the present time as to whether the expense of grouting the strands is warranted, either for the protection of the stressing element or for the increased strength of the beam. I hesitate to draw a conclusion from this one test. As a continuation of our cooperative research program, extensive tests on this subject are to be made in the near future at Massachusetts Institute of Technology, making use of beams of various ratios of depth to width, beams with strands straight and strands parabolic, beams with strands bonded in poured concrete, unbonded, and bonded by grout. The results of these tests should be conclusive.

The costs on the Danvers job were not quite comparable with those for a composite steel stringer design, but were close enough to encourage further designs. There is a considerable saving in the amount of critical materials in a bridge of prestressed concrete design. The nine designs we have made, including Danvers, indicate that about one-third as much steel is required as in a composite steel deck. This point is important, particularly in a period of shortage of critical materials.

Finally, this prestressing project has been a delightful experience. The manufacture and stressing of these beams turned out to be comparatively simple operations, and no one should have any hesitation or uncertainty about building a prestressed bridge.

Grateful appreciation is expressed to Prof. Myle J. Holley, Jr., A.M. ASCE, and Audux Ofjord of Massachusetts Institute of Technology; to A. L. Delaney, A.M. ASCE, of the Portland Cement Association; and particularly to William F. Callahan, Commissioner of the Massachusetts Department of Public Works, without whose sympathetic understanding and cooperation our prestressed program would not have been started.

(This article is based on a paper presented by Mr. Rundlett at a joint session of the ASCE Construction Division and the American Concrete Institute, presided over by A. E. Cummings, chairman of the ASCE Research Committee, at the Chicago Centennial Convention.)

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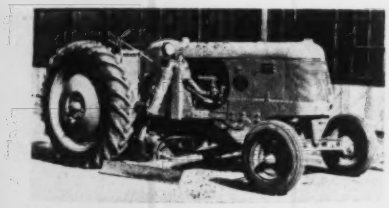
EQUIPMENT, MATERIALS and METHODS

NEW DEVELOPMENTS OF INTEREST AS REPORTED BY MANUFACTURERS

Side Dozer

THE HUBER SIDE dozer is mounted on the firm's 42½ hp maintainer and is designed for scalping beams and shoulders under guard railings, a job which heretofore has been done by hand operations. The side dozer is hydraulically powered for removing sod, gravel and other accumulated debris under railings. It is mounted on the blade suspension system



Designed for Scalping Under Railings

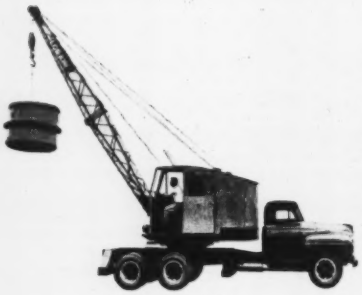
of any existing Huber maintainer. The cutting blade is made of high carbon steel, is 48 in. long, 6 in. high, and a ½ in. thick, and is designed to scrape cleanly and efficiently under any guard railing clearing the ground by 6 in. Maximum reach of the side dozer is 72 in., and it weighs approximately 520 lbs. The overall width of the maintainer with side dozer in retracted position is 96 in. The side dozer becomes the 10th attachment for the Huber maintainer, which is basically a road maintenance machine with standard blade. Other attachments include: bulldozer, lift loader, berm leveler, road planer, highway mower, patch roller, broom and one-way and two-way snow plows. **Huber Manufacturing Co., Marion, Ohio**

Electronic Tool

A PORTABLE ELECTRONIC instrument for locating sources of trouble in all types of construction equipment has just been announced. Known as the Elec-Detec, this electronic stethoscope saves time, work and trouble for maintenance men by locating friction noises in bearings, pistons, gears, ratchets, cams, clutches, and other parts. The instrument uses a metal probe which serves as a microphone to locate the exact source of tell-tale noise. Sound impulses are transmitted through an amplifier to headphones. The Elec-Detec helps to diagnose the trouble and determine quickly where to make repairs without tearing down the entire equipment. Sounds can be detected at low speed that otherwise would be heard only at high speeds. Elec-Detec is furnished complete with batteries and leather carrying case. **Anco Instrument Div., 4254 West Arthington St., Chicago 24, Ill.**

Truck Crane

THE ADDITION of a 6-ton truck crane, the Model TL-10, to the company's line of products, has been announced. The TL-10 crane consists of a complete superstructure equipped as lifting crane which can also be used as a ⅜ yd dragline or clamshell. It is designed primarily for field mounting on a suitable new or used truck furnished by the customer. However, it has many other applications such as mounting on piers, barges, bins, ships docks, trailers, flat cars and the like. It incorporates all of the "balanced quality" features of the well known Lorain TL-Series such as unit assembly of packaged components, oil enclosed gears, anti-friction bearings, interchangeable clutch shoes, safety glass windows, lights and the smooth, fast operation for which the Lorain "TL" has always been noted. The TL-10 is a two drum, gasoline-powered machine fully equipped with a 25 ft, 2-piece butt-flange connected



Model TL-10

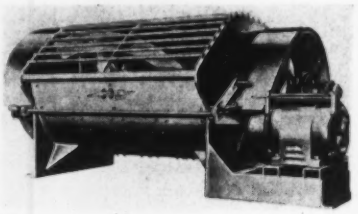
boom. Center sections can be had to extend the boom to 45 ft. Tagline for clamshell service, fairlead for dragline service, precision power boom lowering device and other extras are available also. **The Thew Shovel Company, Lorain, Ohio**

Three-Drum Hoist

AN IMPROVED TYPE of three-drum hoist has been developed especially for operating the company's rapid-shifting drag scraper machines. In general design features, the hoists resemble the Sauerman standard roller-bearing scraper hoists. Drum shafts are stationary and are pinned to the side stands. Drum and gears are mounted on Timken roller bearings. The clutches are positively pushed into and pulled from engagement through double tapered bearings, no springs being used. Drive is through a silent chain, totally enclosed in an oil tight case. Brakes are on opposite ends of drums from the clutches, thus eliminating heating of one by the other. **Sauerman Bros., Inc., 552 S. Clinton St., Chicago, Ill.**

Batch Mixer

A 75 CU FT batch mixer makes it possible for operators using modern high production block machines to give lightweight aggregates the necessary extra mixing time and still keep their production at full capacity. Since there is no increase in height, the big "75" will normally replace a 42 or 50 cu ft mixer now in use without expensive alterations. Powered by a 50 hp



75 cu ft

electric motor, the 75 cu ft mixer has an air operated discharge door and Mix-Timer as standard equipment. **Gocorp, Adrian, Mich.**

End-Loader

A HYDRAULIC END-LOADER for all makes of motor trucks is announced. Known as the Galion hydraulic "Load-evator," the unit is available in 4 models with loading space of 28 by 84 in. to 34 by 90 in. Models are adaptable to all trucks of 1½ ton size and over and are suitable for the use of virtually every type of business and industry. Load-evators have a load capacity of 2,000 lbs. Among the advantages are: a 50 percent reduction in delivery time on heavy or bulky articles, elimination of the need for costly loading docks, reduction of damage to goods and a lessening of driver fatigue to an important degree. Extensive tests also indicate that, by using Load-evators, the average truck operator can do more work with fewer vehicles and reduced personnel. Power for Load-evator operation is supplied direct from the truck engine through a transmission-mounted power take-off. The latter activates a Galion hydraulic hoist which lifts or lowers the end gate as desired. The Load-evator action is controlled by a single spring-loaded lever which assures constant, positive, safe operation. Heavy tubular lift arms with built-in overload capacity are another important safety factor. In addition to standard models, Load-evators are furnished with a number of special accessories for various industries. These include side loading ramps, support racks for gas and air cylinder deliveries, racks for double-decking, etc. **The Galion All Steel Body Company, Galion, Ohio.**

Equipment, Materials & Methods (Continued)

Diesel Pile Driving Hammer

A REVOLUTIONARY SELF CONTAINED diesel pile driving hammer is being placed on the market. The hammer's chief advantages are the elimination of a steam or air generating unit, its labor saving features, high mobility, reduced setting up time, fuel economy and low initial cost. Due to the hammer's compact arrangement, a pile driving crew may be reduced to an operator and one helper, thus saving about 50 percent in labor. Its shock free characteristic permits light weight tubular leads to be used, which may be mounted on a tractor, crane or barge, with resulting quick movement to job sites. The initial model which is of medium capacity is particularly suited to average and small jobs such as the construction of boat landings, piers, docks, revetments, foundations for buildings, and railroad and highway maintenance work. Wood, concrete and steel piles and sheet piling can be driven economically up to 2 tons in weight. An average wood pile can be driven to a 150 ton load bearing capacity at a rate up to 2 ft per min. The operation of the hammer is started after the pile is lifted to a driving position and secured to the leads. With the driving anvil and the body of the hammer resting on the pile the heavy ram shaped piston is raised to its starting position by a line from a small hoist. When the piston reached its upward position the cocking mechanism is disengaged. During its downward travel a fuel injection pump cam is actuated, thus injecting fuel into the lower combustion chamber. As the piston nears the lower part of the cylinder the exhaust ports are closed off and compression is started. When the piston strikes the driving anvil atomization of the fuel under a pressure of over 500 lbs per sq in. is accomplished. This model which is of medium capacity delivers 7500 ft lbs per stroke at the rate of 60 strokes per min. Overall dimensions are 14 in. in diameter by 9 ft, 10 in. long. **MTP Company of America, 14031 Huston St., Sherman Oaks, Calif.**

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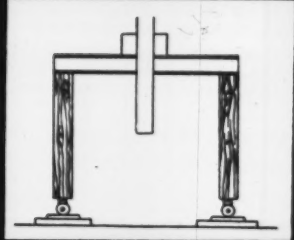

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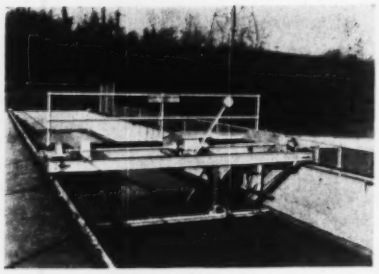
THE MARKETING of the Butane fired salamander, The Port-A-Heat, has been announced. Offering greater safety, longer life, and less fire hazard, Port-A-Heat salamanders are especially designed for contractors, plasterers, paper hangers, floor covering workers, brick layers and carpenters. The unit is smokeless, greaseless, sootless and non-toxic. This permits operation without the use of a vent pipe. Unlike coke or oil burning salamanders the Port-A-Heat unit does not get red hot—consequently it may be easily moved at will without waiting for metal hood or sides to cool. The unit weighs only 14 lbs, is 21 in. high and 13 in. wide. Output up to 36,000 B.T.U. per hour. Finish is galvanized. It is furnished with 5 ft armoured hose for tank connection, 20 ft hose length is available. L. P. gas tank is not furnished with unit. **Port-A-Products Company, P. O. Box 515, Far Hills Station, Dayton, 9, Ohio.**

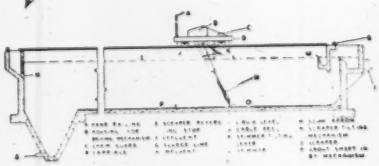
Load Packer

A LOAD-PACKER, featuring 15 major improvements for more sanitary and lower-cost refuse collection, has been announced. Capacities of the three new models have been increased to 10, 13, and 16 cu yds. These increased capacities have been engineered to maintain minimum wheel base requirements for greater maneuverability in narrow alleys. In addition to the larger rated capacities, the Load-Packer insures even greater loads and fewer trips to the disposal point by more effective compaction which exerts a more direct line of force against the material being compressed into the body. It also features a fully automatic compacting cycle, operated by a single finger-tip control. The ram opens ready for reloading automatically, enabling crew men to make the next pick-up during the compacting cycle. A hydraulically controlled retaining panel insures positive retention of all materials in the Load-Packer body. The wider, larger loading hopper provides ample space for crews to load from both sides simultaneously with no delays or interference. The hopper is also lower, and requires less physical effort to lift and dump heavy refuse container, enabling pick-up crews to keep a constant pace all day. The streamlined Load-Packer has a double tubular seal between the tailgate and body which eliminates odorous and bacteria laden fluids from dripping onto the street. An optional sump tank which separates and disposes excess liquid drained for the refuse being compacted is available at no extra cost. Gar Wood is offering the hoist and body as a "packaged unit" which the manufacturer says will speed-up and reduce costs of installation. The Load-Packer can be installed on any type truck chassis, including tandem axle units. **Customer Service Department, Gar Wood Industries, Wayne, Mich.**



RECTANGULAR CLARIFIERS

FOR SEWAGE AND TRADE WASTES



Particularly suitable where space is limited or where sludge delivery is desired at one end of tank. Consists of a bridge crane spanning width of the tank, supporting sludge scraper and skimmer, which moves automatically back and fourth.

Many of these units are now operating in sewage plants throughout the country.

This unit has also proven highly successful as an oil-water separator in the treatment of oil refinery waste water.

Bulletin 31-D-37.

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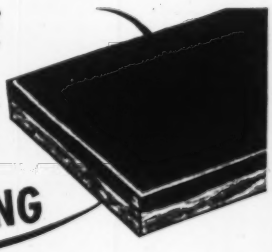

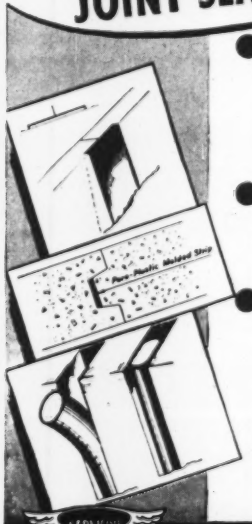

PARALATERAL Wide Strip
For sealing vertical construction or expansion joints in Retaining Walls, Abutments, Wing Walls, Foundations, etc., particularly when one side will be backfilled and protection from water seepage is necessary. Has rigid backing of asphalt joint material and surface and both edges coated with Para-Plastic, which maintains bond with concrete at all times.

MOLDED STRIP
Para-Plastic Sealing Compound is molded into strips for sealing of keyed construction joints and cracks or breaks in vertical concrete surfaces. Concrete poured against the strip, after setting up, will bond with strip to form watertight seal.

NARROW STRIP Pressure Seal
Para-Plastic Pressure Seal is available in both Solid Para-Plastic, and Para-Plastic coated sponge rubber types which are rectangular or square in cross section. Designed for sealing existing expansion joints or contraction cracks in vertical or sloping masonry walls and surfaces.

Servicised Engineers are available for consultation on your joint sealing problems. Write for details and specification data. There is no obligation.

*Para Plastic is one of the many patented products developed for the Construction Industry by Servicised Products Corp.

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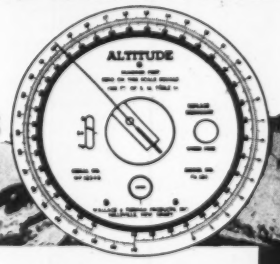

Whether you're surveying rough terrain or gentle slopes, it's nice to know that your precision altimeters are rugged and durable enough to withstand severe shocks and vibrations. This security is yours with W&T Sensitive Altimeters.

Consider particularly the W&T Sensitive Altimeter Type FA-181 — an instrument in which construction and precision meet the exacting requirements of military service. The Type FA-181 is aluminum cased and has a latched metal lid for protection under all climatic conditions. Several standard ranges are available. The -1000 to 6000 foot range, for example, is readable to one foot — with sensitivity of one part in 8000. A desiccant (with a condition indicator) is included to absorb moisture which may enter the case. The Altimeter is compensated for temperature changes and readings do not require correction.

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Prepared under the direction of the Committee on Engineering Practice

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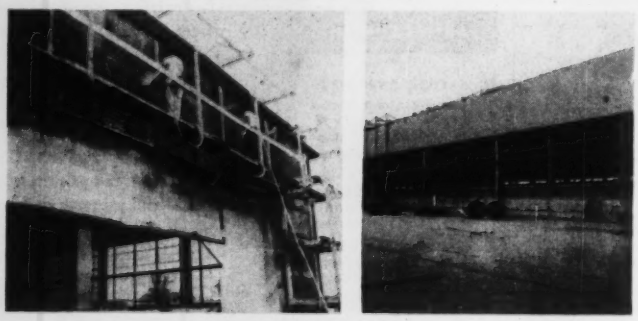

CEMENT GUN COMPANY APPLIES "GUNITE" ON WALLS OF NEW FACTORY

Photo, at left, shows our men applying 2" mesh reinforced "GUNITE" on outside wall during the construction of a new 1500' x 600' factory in Delaware. Photo, at right, shows a large area of completed work.

This project was done under plans and specifications of Albert Kahn Associated Architects and Engineers who have specified hundreds of thousands of square feet of this type of "GUNITE" wall work for Ford, Chrysler, General Motors, Pratt and Whitney, Glenn Martin and other large manufacturing concerns.

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Equipment, Materials & Methods (Continued)

Heavy-Duty Dozer

A HEAVY-DUTY dozer to handle tougher and bigger jobs than ever before possible in a dozer of comparable cost and size, is being introduced to loggers, contractors, industrial users and farmers. Improved design and use of a new alloy, Ductilite, an International Nickel Corporation research development, reduces weight at vital balance points while holding to the strength

of steel. Actual weight reduction in the frame is close to one-fifth with resultant lessening of strain on the tractor itself. Because of its balanced, work-engineered construction, the Holt heavy duty will cut upkeep and operation costs and actually prolong the life of the tractor. The design also permits use of the same dozer frame with tractors of different track sizes. The model has extra high lift and versatility and is controlled by an improved finger-tip control hydraulic system that may be used for other equipment when the dozer blade is disconnected. **Independent Distributors, 27 N.E. Broadway, Portland, Ore.**

Plastex Pipe

A YELLOW PLASTIC pipe which can be easily seen and quickly identified is now being marketed. The exclusive coloring of Plastex yellow pipe has been especially compounded by Plastex engineers for maximum visibility and durability. The color, which goes all the way through the plastic material, will not fade or stain under any conditions. For mining operations, the pipe has a definite advantage, since lines laid in darkened passages are easily distinguished eliminating the danger of being damaged by other equipment. Another safety feature of the pipe is that it can be positively traced and identified when used with other pipe lines or cables. The pipe has all the advantages of lightweight, flexibility and guaranteed service against rust, rot and corrosion found in other types of Plastex pipe now being used for many industrial, farm and home applications. **The Plastex Pipe & Extrusion Co., Columbus, Ohio**

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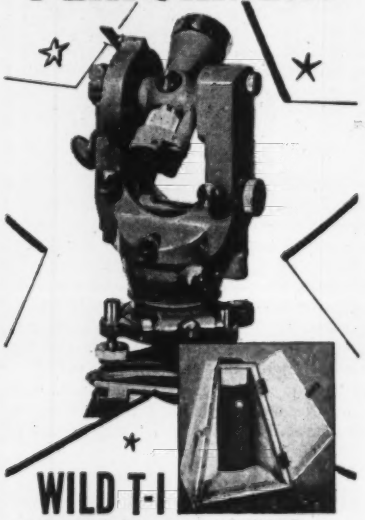

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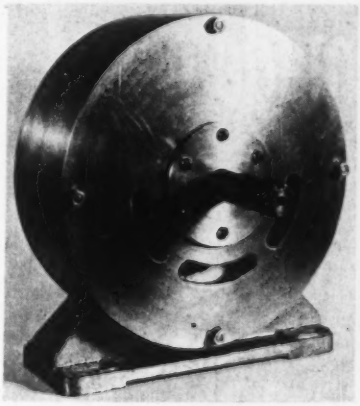
Equipment, Materials & Methods (Continued)

Calculating Machine

A CALCULATING MACHINE called Coorapid has been announced. This instrument which is patented world wide has aroused the interest of surveyors, oil prospectors, mining engineers, architects, contractors, aircraft industries, universities, forest and agricultural departments, the Bureau of Reclamation, and other departments of states, cities, and the Federal Government. Coorapid works a hundred times faster than any calculating machine on the market. It does not require highly paid specialists for its handling but can be operated by anybody after a few hours' training. Coorapid does not actually require calculation: distance and angles are set on this instrument and all pertinent data can be read from it instantaneously. **Geo-Optic Company, Inc., 170 Broadway, New York 38, N. Y.**

Electric Generator

THE IMPROVED "Nobrush" electric generator, featured in a 400-cycle type, avoids brushes entirely by a permanent magnet construction. Among the advantages attained by this construction are: immunity to damage by moisture; immunity to damage by overload or short-circuit; durability and long life; reduction



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in size and weight; almost total elimination of maintenance; and elimination of radio interference. Units are furnished for belt drive or for mounting on driving motor or engine. Complete motor-generator or engine generator sets are available. The generator is adapted for field supply of 400 cycles, for portable power units, for mounting in marine craft or vehicles, or for operation of newer high frequency portable power tools. Outputs range from 250 VA to 25 KVA. Usual voltages, single or three phase are provided. **The Georator Corporation, Electric Products Division, Arlington 9, Va.**

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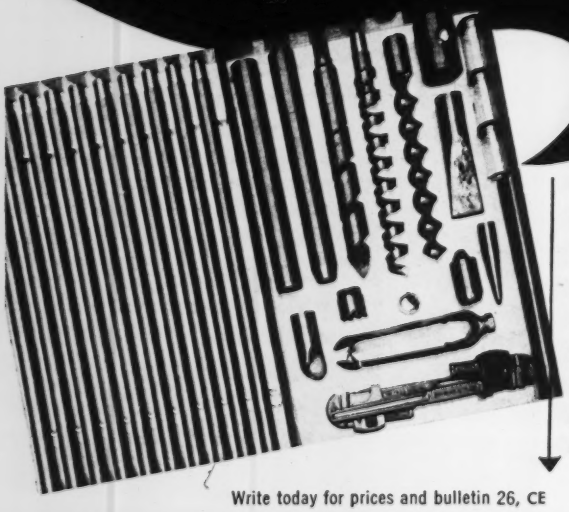

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Literature Available

HEAVY EQUIPMENT—Moving big loads over long hauls at high speeds constitutes the type of work presented in "Big Producer," a booklet recently published. The title of the booklet is used to denote the "Cat" DW21 tractor with No. 21 scraper. This four-wheel combination is described as having a maximum speed of 20 mph, a heaped capacity of 20 cu yds and a 225 hp rating. **Caterpillar Tractor Co., Peoria 8, Ill.**

SAFETY TREAD INFORMATION—A file size folder is now offered to provide detailed information quickly to engineers, architects and designers on practically every type of safety tread application. It contains 28 plates of details on abrasive cast and safe groove treads as well as expansion plates, platforms, curb bars and floor grids. Full size cross-sections of the various types are shown in addition to dimension drawings, typical installation and mechanical specifications. **Wooster Products, Inc., Dept. C. 102, Wooster, Ohio**

JOINT SEALING COMPOUND—A revised, 4-page data sheet, No. I-H 601, describing Flintseal rubber asphalt hot-poured joint sealing compound for concrete pavements has been issued. Illustrations show the latest specialized equipment used for melting and pouring Flintseal and also methods and machines used in cleaning and preparing joints for sealing. **The Flintkote Company, 30 Rockefeller Plaza, New York 20, N. Y.**

DIESEL ENGINES—Publication of 12 two-page specifications sheets on the Type 4FS one, two and three cylinder diesel engines is announced. These bulletins give complete engine specifications, equipment data and outline drawings of the various engine models. **Nordberg Manufacturing Company, Milwaukee 7, Wis.**

GENERAL CATALOG—A 16-page catalog on "Basic Units" for stationary crushing, screening and washing plants has been issued. The bulletin explains the purpose, features and specifications of Pioneer crushers, feeders, conveyors, vibrating screens, revolving screens, scrubbers, dehydrators, bins and related units. Of particular interest to quarry, mine and cement plant operators are several pages of detail drawings covering stationary primary units. **Pioneer Engineering Works, Inc., 1515 Central Ave., Minneapolis 13, Minn.**

SLIDE RULE—The practical application of mathematical principles is deftly handled in a 120-page self-instruction text just published. The text is divided into three distinct sections and deals with the improved principles of the Versalog slide rule as it applies to electrical, mechanical and civil engineering. Each section presents a practical, easy to comprehend guide to the efficient use of the Versalog in these specialized fields. **Frederick Post Company, Dept. TX, 3650 N. Avondale Ave., Chicago 18, Ill.**

TIDE GATES

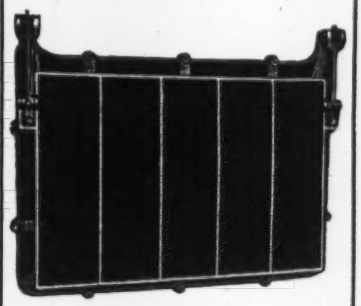

Fig. B-61. Type M-M

Type M-M (Rectangular) Tide Gates are available in 37 sizes from 8″ × 8″ to 96″ × 96″. Bulletin No. 71 describes them fully.

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Literature Available (Continued)

RUBBER ROADS—A 52-page booklet entitled, "Stretching Highway Dollars With Rubber Roads," is offered. This booklet gives the whole story on the history of rubber roads, and describes in detail the test roads using natural rubber powder that have been laid in the United States and Canada through the 1951 paving season. **Natural Rubber Bureau, 1631 K Street, N. W., Washington 6, D.C.**

PUBLIC UTILITY BROCHURE—An attractive and informative brochure entitled "A Study in Progress" is now available. Prepared for the public utility industry, it is the unusual case history of how one major public utility is meeting the problem of reducing costs and increasing worker productivity in a phase of operation important to all—handling, storing and warehousing the huge volume of supplies and equipment essential to day-by-day utility functioning. Featured are a great many photos of industrial trucks in actual operation. **Hyster Company, 2902 N. E. Clackamas St., Portland 8, Ore.**

WIRE ROPE HANDBOOK—The fact-packed, 64-page "Wire Rope Handbook" helps users become more expert wire rope buyers. It contains valuable, every-day information in a handy size booklet. The contents include descriptions, diagrams and illustrations of wire rope types and constructions, as well as helpful information about lubricants, working loads, safety factors, and specifications. Forty separate charts give breaking strengths and weights of wire rope by constructions and sizes, along with calculations for the proper selection of wire rope attachments. **Dept. R-51, A. Leschen & Sons Rope Co., 5909 Kennerly Ave., St. Louis 12, Mo.**

HARD-FACING GUIDEBOOK—The rebuilding and hard-facing of all types of heavy equipment used in earth-moving, mining, lumbering, cement and allied industries is today considered almost standard procedure. The various materials used and the methods of applying these alloys are of prime interest to maintenance men. A booklet called "Stoody Hard-Facing Guidebook" has recently been completely revised and is now available to those interested in the subject matter covered. **Stoody Company, Whittier, Calif.**

COMPRESSOR—A catalog covering Model HMA compressors is announced. While these units are primarily designed for the petroleum industry, they are well suited to heavy construction projects using bottled propane as a fuel. In this role, they will replace a large number of small, high speed units with an extra rugged, moderate speed unit with substantially less maintenance. Detailed information, specifications and engineering data are included. **Clark Bros. Co., Olean, N.Y.**

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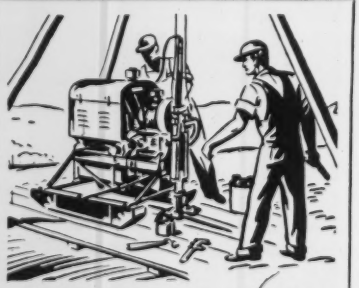

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Literature Available (Continued)

FOUNDATION PIPE—A revised edition of a booklet on foundation pipe, covers pipe shells, pile shells, and caissons. It deals with sizes, end preparation, mill service, field advantages and contains specifications and tables on dimensions and properties of spiral welded pipe piles as well as data on Hel-Cor pile shells. **Armco Drainage & Metal Products, Inc., Middletown, Ohio**

ASPHALTIC PRODUCTS—"Laykold For Cold Insulation Construction" is the title of an 8-page, two-color booklet. With detailed charts, photographs and tables plus complete, easy-to-read descriptive material, the booklet provides valuable data on typical applications of Laykold insulation adhesive, Laykold cement and Laykold weathercoat. **American Bitumuls & Asphalt Company, 200 Bush St., San Francisco 4, Calif.**

MOTOR GRADERS—"Controlled Earthmoving" is the title of an 8-page publication on motor graders. There are a dozen on-the-job photos of the three sizes of Cat motor graders, Nos. 12,112 and 212. The action pictures enumerate some of the many construction uses on highways, dams and airports. **Caterpillar Tractor Co., Peoria, Ill.**

INDUSTRIAL TRACTORS—The 75 percent fuel costs savings of Sheppard full diesel power is stressed in a booklet on industrial tractors. The versatility and maneuverability of the wheel type tractors are illustrated by the company's SDI-3 working with a Lull shoveloader, Hopto digger and special purpose units for mines, quarries and railroads. **Sheppard Diesels, Hanover, Pa.**

SELECTING A CRUSHER—To assist "the man who, sometime, may want to select a crusher," a handbook bearing that title explains in a simple non-technical manner the various crushing methods and where they apply in crushing procedure. Every person who has anything to do with mechanical reduction will want this booklet. **Pennsylvania Crusher Company, 1700 Liberty Trust Bldg., Philadelphia 7, Pa.**

INDUSTRIAL WASTE TREATMENT—An 8-page bulletin "Industrial Waste Treatment Guide" contains a highly useful table showing the different types of combinations of unit treatment processes for more than 50 wastes. A second table shows the equipment used in the several unit treatment processes. A third table shows B-I-F Industries equipment available for use in these waste treatment processes. The bulletin contains a two page spread flow diagram showing how the various treatment processes and equipment may be combined to treat mixed wastes. Installation photos of equipment are also included. **B-I-F Industries Inc., 345 Harris Ave., Providence, R.I.**

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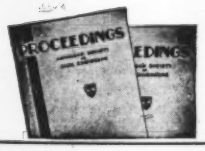

PROCEEDINGS AVAILABLE

The following papers, printed as Proceedings Separates, may be ordered on the basis of summaries given in this and previous issues of CIVIL ENGINEERING. Discussions of these papers will be received, as in the past, for a period of five months following the date of issue. A summary of each paper appears in several consecutive issues; other titles will be added every month, as they become available. Use the convenient order form on page 120.

Summarized in Earlier Issues

148. **Bank Stabilization by Revetments and Dikes,** by Raymond H. Haas and Harvill E. Weller.

149. **Industrial Waste Treatment in Iowa,** by Paul Bolton.

150. **East St. Louis Veterans Memorial Bridge,** by A. L. R. Sanders.

151. **Topographic Mapping in Kentucky,** by Phil M. Miles.

152. **Methods for Making Highway Soil Surveys,** by K. B. Woods.

153. **Characteristics of Fixed-Dispersion Cone Valves,** by Rex A. Elder and Gale B. Dougherty.

154. **A Navigation Channel to Victoria, Tex.,** by Albert B. Davis, Jr.

155. **Field Study of a Sheet-Pile Bulkhead,** by C. Martin Duke.

156. **Rice Irrigation in Louisiana,** by E. E. Shutts.

D-78. Discussion of Paper, **River Channel Roughness,** by Hans A. Einstein and Nicholas L. Barbarossa.

D-109. Discussion of Paper, **Final Foundation Treatment at Hoover Dam,** by A. Warren Simonds.

Third Notice

157. **Radial Impact on an Elastically Supported Ring,** by Edward Wenk, Jr.

158. **Flexure of Double Cantilever Beams,** by F. E. Wolosewick.

D-108. Discussion of Paper, **Control of Embankment Material by Laboratory Testing,** by F. C. Walker and W. G. Holtz.

D-113. Discussion of Paper, **Wave Forces on Breakwaters,** by Robert Y. Hudson.

D-115. Discussion of Paper, **Lake Michigan Erosion Studies,** by John R. Hardin and William H. Booth, Jr.

Second Notice

159. **Development of a Flood-Control Plan for Houston, Tex.,** by Ellsworth I. Davis. This paper examines the problems arising from delay in construction of flood-control plans in one of the most rapidly growing communities in the United States, and outlines steps in revising the flood-control plan. The procedure in developing a project design flood is presented and a comparison made of plans for diversion of floodwaters and for rectification of stream channels to carry the floodwaters. The problems of the disposal of large quantities of excavated material in developed municipal areas are discussed. The author writes of the advantage of lining flood discharge channels under certain conditions. (Available January 1.)

160. **Ice Pressure Against Dams: Studies of the Effects of Temperature Variations,** by Bertil Löfquist. The magnitude of the ice pressure remains as one of the major uncertainties in the design of hydraulic structures. Opinions often differ considerably. However, relatively little has been done to solve this problem—possibly because of the complexity of the task. The author has measured directly the pressures in an artificial ice sheet in a freezing chamber. The results are somewhat contradictory to earlier investigations. The ice pressure does not seem to increase with the thickness of the ice sheet, but a maximum value is found for a thickness of approximately 20 in. The author has arrived at a maximum pressure of from 20,000 lb per lin ft to 27,000 lb per lin ft. These figures are based mainly on calculations of the buckling load in the ice sheet. (Available January 1.)

161. **Ice Pressure Against Dams: Some Investigations in Canada,** by A. D. Hogg. The thick ice and possible high ice pressures in existing and projected reservoirs in Ontario and other provinces necessitate the determination of design values for maximum ice pressures. This paper describes equipment and methods used in this work, and indicates the extent to which such experimentation has been carried on at several Canadian dams. (Available January 1.)

162. **Ice Pressure Against Dams: Experimental Investigations by the Bureau of Reclamation,** by G. E. Monfore. A description of investigations of the pressures produced by thermal expansion of ice sheets is presented for discussion. The construction of equipment and the test procedures are explained. The program included numerous measurements at five reservoirs; and the effects of shore line characteristics and solar radiation have been considered. Laboratory investigations, which verified the field measurements, included studies of the effects of initial ice temperature and rate of temperature rise. (Available January 1.)

163. **Design Methods for Airfield Pavements: Progress Report of the Committee on Correlation of Runway Design Procedures of the Air Transport Division.** The design of airfield pavements has received increased attention since World War II. The methods used in the United States and Canada were studied and compared by the committee. Qualitative comparisons are found practicable on the basis of the relationships between the California

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Bearing Ratio used by the Corps of Engineers and the subgrade strength indices used by the Canadian Department of Transport, the Civil Aeronautics Administration, and the United States Navy. The Westergaard procedure for the design of concrete pavements is summarized and compared with the CAA method. (Available January 1.)

D-69. Discussion of Paper, **Valuation and Depreciation Related to Income Tax,** by Maurice R. Scharff.

D-77. Discussion of Paper, **Buckling Stresses for Flat Plates and Sections,** by Elbridge Z. Stowell, George J. Heimerl, Charles Libove, and Eugene E. Lundquist.

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D-93. Discussion of Paper, **Aircraft Design as Related to Airport Standards,** by Milton W. Arnold.

D-97. Discussion of Paper, **Consumptive Use in the Rio Grande Basin,** by Robert L. Lowry.

D-98. Discussion of Paper, **Consumptive Use of Water on Irrigated Land,** by Wayne D. Criddle.

D-99. Discussion of Paper, **Consumptive Use in Municipal and Industrial Areas,** by George B. Gleason.

D-101. Discussion of Paper, **Application of Highway Capacity Research,** by J. P. Buckley.

D-102. Discussion of Paper, **Utilization of Ground Water in California,** by T. Russel Simpson.

D-103. Discussion of Paper, **Pile Foundations for Large Towers on Permafrost,** by L. A. Nees.

D-105. Discussion of Paper, **Principles of Highway Capacity Research,** by O. K. Normann.

D-112. Discussion of Paper, **Diversions from Alluvial Streams,** by C. P. Lindner.

First Notice

164. **Water Supply Engineering, a Report of the Committee on Water Supply Engineering of the Sanitary Engineering Division for the Period Ending September 30, 1951.** Current trends in all phases of water supply are described in this report. Some of the many items reported on are: The artificial recharge of ground water; the effects of droughts, such as that in New York, N.Y., in 1949; the 1951 floods in the midwestern states; the design of dams, tunnels, pipelines, pumping stations, and treatment works; and the construction of many new water supply projects. The latest treatment methods are described, and recent investigations and reports are reviewed. (Available February 1.)

165. **Design Curves for Anchored Steel Sheet Piling,** by Walter C. Boyer and Henry M. Lummis, III. The authors are cognizant of the work encountered in many pier development projects in the preliminary design stage. Frequently one must develop a number of sheet pile sections for varying conditions of surcharge. A set of curves introduced with this paper will simplify such preliminary design calculations and in many cases will suffice for the final design as well. (Available February 1.)

166. **The Design of Flexible Bulkheads,** by James R. Ayers and R. C. Stokes. The wideness of variety among theories and mathematical studies on the action and effect of earth pressure on flexible bulkheads makes discussion of the practical considerations particularly valuable for designers. The authors have presented for consideration by the profession, a description of procedures used by the Bureau of Yards and Docks, U. S. Navy. They have explained soil pressure theories, presented pressure and loading diagrams, and described methods for solving the problems arising in the construction of sheet pile bulkheads. (Available February 1.)

167. **Sewage Disposal in Tidal Estuaries,** by Alexander N. Diachishin, Seth G. Hess, and William T. Ingram. The estimations of dilution volumes and detention periods for sewage discharged into tidal waters are often within the province of the sanitary engineer. This paper presents information pertinent to these estimations about the types of tides that can occur in estuaries and the influence of salinity, winds, and changes in atmospheric pressure on tidal flows. Methods used in calculating dilution volumes and detention periods are discussed with the purpose of determining how well these methods are in accord with the accepted concepts of tidal flow. (Available February 1.)

168. **Special Design Features of the Yorktown Bridge,** by Maurice N. Quade. The George P. Coleman Memorial Bridge spans the York River at Yorktown, Va. At its center are two swing spans placed in tandem, each one longer than any previous swing span built. The shore end of each swing span rests, in the closed position, on the end of a 140-ft cantilever arm. The design of special caissons for the piers and of special connecting and wedging apparatus for the swing spans involved several unusual problems whose solutions are presented. (Available February 1.)

D-117. Discussion of Paper, **The Development of Stresses in Shasta Dam,** by J. M. Raphael. This paper, published in February 1952, described the results of long-term observations of the structural behavior of Shasta Dam, made possible by instruments embedded in the concrete. Discussers are: Ross M. Riegel, Roy W. Carlson, J. Laginha Serafim, A. D. Ross, J. A. Hanson, A. Warren Simonds, and J. M. Raphael. (Available February 1.)

D-122. Discussion of Paper, **Engineering Aspects of Diffraction and Refraction,** by J. W. Johnson. The original paper, published in March 1952, is a study of the phenomena of refraction and diffraction and their connection with engineering solutions to shoreline problems. Discussion by: M. E. Stelzriede and J. W. Johnson. (Available February 1.)

D-125. Discussion of Paper, **Torsion of Plate Girders,** by F. K. Chang and Bruce G. Johnston. The original paper, published in April 1952, describes analytical and experimental research for the purpose of relating rivet pitch or weld size to the torsional stiffness and strength of built-up members. Discussion by: Arthur P. Jentoft, Richard W. Mayo, and E. Russell Johnston; and F. K. Chang and Bruce G. Johnston. (Available February 1.)

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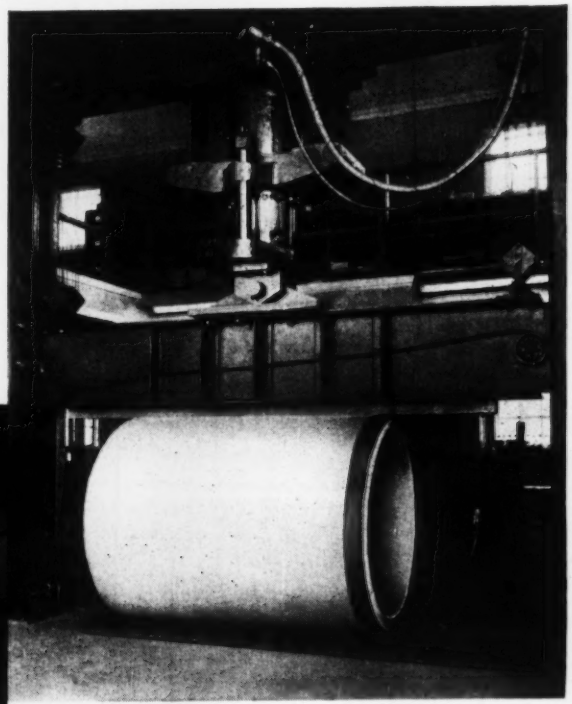

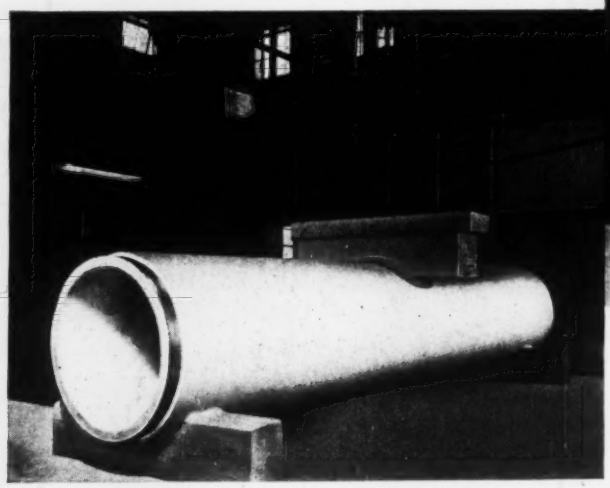

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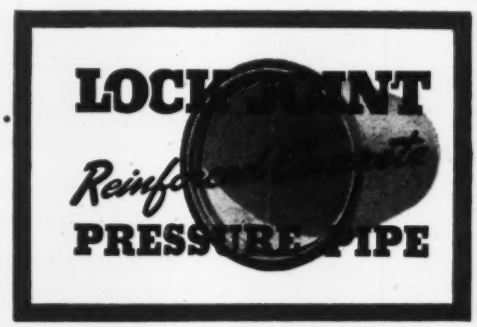
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